TRISHNAAH
quench your thirst of knowledge

Journal for Recent advances in Built Environment (JRABE)

ISSN: 2583-3901

In Association with Department of Civil Engineering
K.S, School of Engineering and Management

Volume 2 2023

Climate responsive buildings for Hot and Dry climates

Dr.Vijayalakshmi Akella [1]

1. Professor and Head : Department of Civil Engineering, K S School of Engineering & Management, Bangalore, India

zabfathimaks@gmail.com

1 INTRODUCTION

Indian vernacular architecture originated from the functional point of view considering the region's climatic conditions. India has predominantly six climatic zones namely, hot and dry, warm humid, hot and humid, cold, composite and moderate climate. The architecture in these regions is unique in nature as local materials were used to build these structures. The temperature is in the range of 40- 47 o c and humidity 30-50% in hot and dry climate zones. This kind of condition is prevalent in the cities of Jaipur; Hyderabad etc. These cities are also exposed to dust storms hence have small openings in their structures.

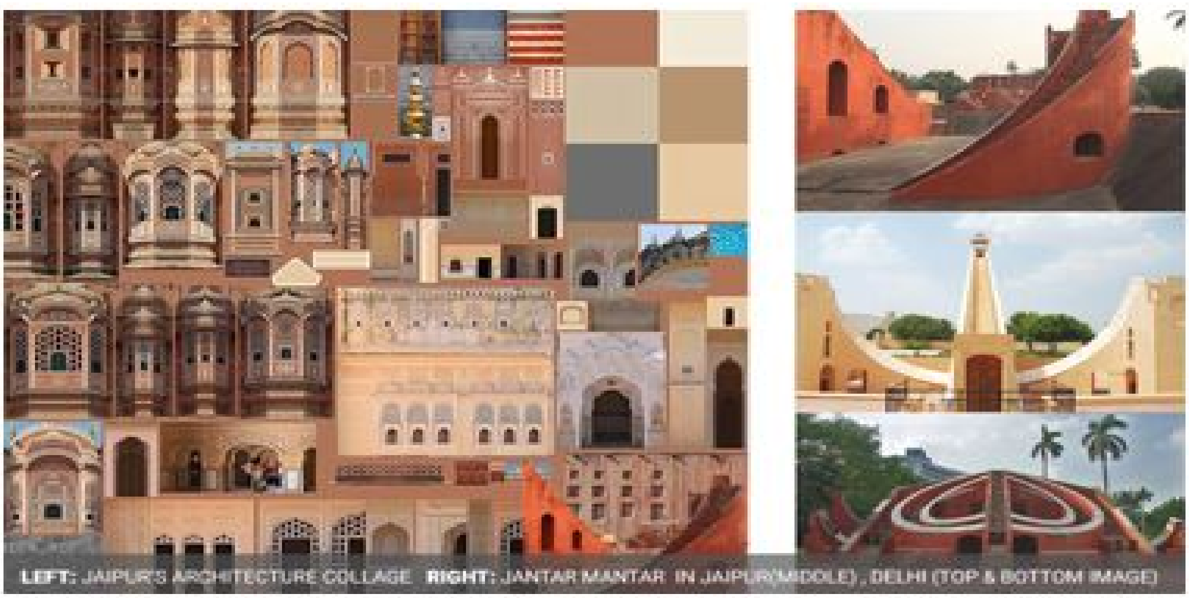


Figure 1: Dwelling in Jaipur with small openings Source: Google Images

The Jaali work in the openings allows cool breeze inside the buildings as in case of Hawa Mahal. It looks similar to beehive in the honey comb. Most of the structures use pink and red sand stone which is locally available. The orientation of the buildings plays an important role in these climatic zones. The long walls facing North and South direction is the most preferred orientation. The kitchen should be located on leeward side of the building to avoid circulation of hot air and smell from the kitchen. The Air movement rate in indoor spaces increases the cooling efficiency in hot and warm seasons. Natural ventilation is one of the passive design strategies, which enhances indoor air quality in hot and dry regions by providing fresh air. This in turn reduces the energy bills. The shading of roof's, windows is an essential part of hot and dry climates. Vegetation is an important aspect to be considered while designing buildings in this climate zone.

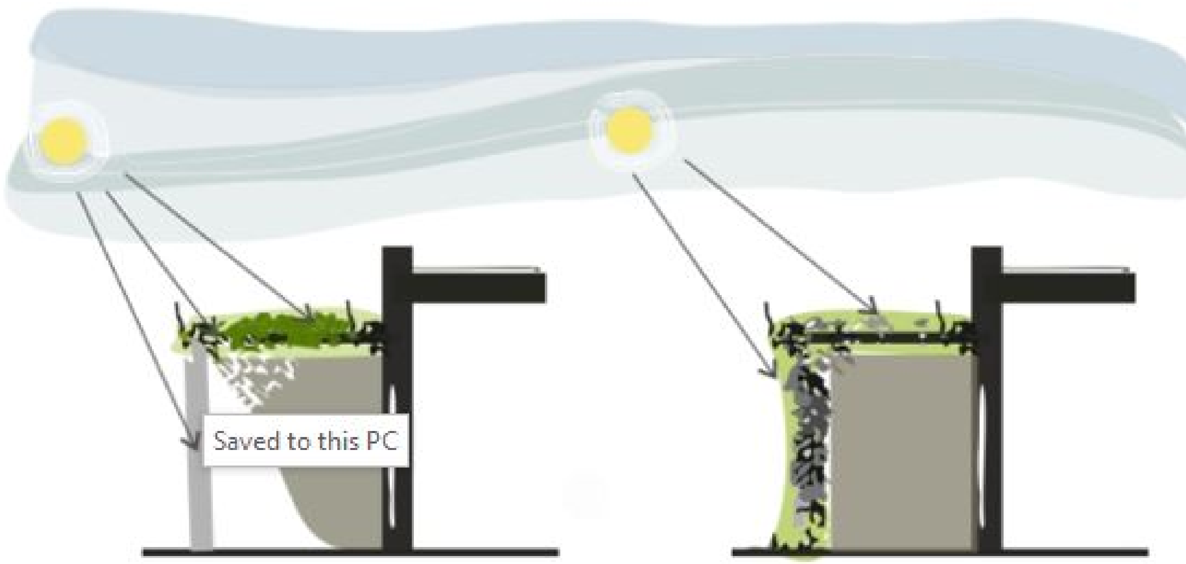


Figure 2: Shading Methods

Evaporative cooling reduces the indoor temperature. A water body in the courtyard or in the way of wind flow can be beneficial in cooling the interiors. The design methods adopted in traditional vernacular architecture are noteworthy. These techniques should be adopted while designing instead of aping the contemporary architecture elsewhere. Buildings in this zone should not use glass façade though there is an advance in the manufacturing of glass. Glass with low U values are available in the market but still should be avoided in these zones as reflection will be more hence heats up the surroundings. Buildings should also be spaced closer so that mutual shading of buildings takes place and keep the indoor environment cool.

Design and development of a finite-element model for the Amritesvara temple: A dry-stone heritage structure

Vasantha Lakshmi Gudasali [1], Dr Vijayalakshmi Akella [2], Dr B. K. Raghuprasad[3]

1. Research student - Department of Civil Engineering, K S School of Engineering & Management, Bangalore, India

2.Professor and Head : Department of Civil Engineering, K S School of Engineering & Management, Bangalore, India

3. Retired Professor IISc, Bangalore 560012, India

vasanthalakshmi07@gmail.com

Abstract

India has a vast cultural heritage; hence the assessment of the structural safety and seismic susceptibility of historic masonry structures is a critical task for preserving these structures against unexpected seismic events. Large-magnitude earthquakes in India, such as the Bhuj earthquake in 2001 (magnitude: 7.7), Killari earthquake in 1993 (magnitude: 6.4), and Jabalpur earthquake in 1997 (magnitude: 6.0), have exposed the vulnerability of architectural heritage structures in India. This study investigates the Amrutesvara temple, which is a historic stone masonry temple that is located in south India and protected by the Archaeological Survey of India. A finite-element (FE) model was developed for the Amrutesvara temple, which represents the Hoysala architectural style, according to the results of visual damage assessment, free vibration tests, and a shake table test. The heritage building was analysed under its self-weight. The dynamic properties of the temple were determined from the developed FE model. The natural frequency of the temple obtained through a free vibration test was used to validate the FE model. The fundamental frequency of the FE model with rigid joints was computed to be 1.46 Hz from the obtained eigenvalue solution. Whereas experimentally it was found to be 1.10 Hz. Many cracks were observed in the temple structure. The joints have been subjected to deformations, as is evident from a visual inspection at the site. Consequently, the stiffness of the structure have reduced. The results of this study will help in designing possible retrofitting measures for strengthening various elements of the temple structure.

Keywords

Stonemasonry structure, heritage building, ambient vibration test, finite-element analysis, seismic analysis, ANSYS.

1 Introduction

Stonemasonry is a prehistoric construction technique, and motorless joints played a key role in the construction of many heritage and historical structures. A certain type of heritage stone masonry structure constructed using motorless joints is observed in most parts of south India (Hoysala architecture). This type of structure is characterised by open and closed mandapas, highly ornate mandapa ceilings with circular domes, and lathe-turned pillars. The Hoysala architecture is influenced by the Dravidian style generally found in the Southern Deccan Plateau region. Masonry temples constructed during this period (1026–1343), such as the Chennakesava temple (1117), Hoysaleswara temple (1120), Amrutesvara temple (1196), and Kesava temple (1268), are examples of the Hoysala architectural style. These temple complexes are proposed to be listed as UNESCO world heritage sites (Figures 1 and 2). The heritage structure investigated in this study is the Amrutesvara temple (Figure 3), which was constructed in 1196 and is protected under the Ancient Monument and Archaeological Sites and Remains Act, 1958. The temple is located in Amruthapura village, which is 260.8 km from Bengaluru. The salient feature of the Hoysala temple architecture is the type of stone used for construction. Temples with the Hoysala architecture were constructed using chlorotic schist, which is also known as soapstone. Chlorotic schist is a fine-grained rock, which is malleable and ductile when extracted from the mine. It becomes harder on exposure to the sun. Soapstone was used for the construction of 90% of the Hoysala architecture. The restoration of the Amrutesvara temple was performed previously through conventional methods by supporting the cornice by placing stone columns (Figure 4). No stress analysis has been performed. Moreover, the accurate distribution of stresses and their critical locations are unknown. Cracks and chipping of stone are noted at the beam-column joints (Figure 5), and gaps are observed between the orthogonal walls of the corner.

The aim of the present investigation was to determine the ability of the Amrutesvara temple to survive an earthquake. Historic stone masonry structures are generally fragile and in a poor state; hence, these structures are likely to be unsafe during earthquakes. Moreover, their dynamic characteristics have not been established. Therefore, this paper presents the essential features of the Amrutesvara temple, such as its fundamental frequencies and mode shapes, so that its dynamic response can be evaluated.

2 Objectives

The objectives of this study were as follows:

1. To study the overall structural model of the Amrutesvara temple, including the properties of the materials used in the construction of the monument.

2. To develop and analyse a finite-element (FE) model of the temple.

Figure 1: Chennakeshava temple, Belur, India

Figure 2: Hoysaleshwara temple, Halebidu, India

Figure 3: Amrutesvara temple, Terikere, India

Figure 4: Placement of stone columns for supporting the components of the Amrutesvara temple

Figure 5: Cracks and chipping of stone at the beam-column joints

3 Literature Review

A three-dimensional (3D) FE model was developed using shell elements for masonry walls and frame elements for the column and timber slab members. Jetson et al. [1] modelled south Indian temple structures under earthquake loading. They conducted ambient vibration tests to determine the natural frequency of the mandapam. The obtained experimental results were used to calibrate FE models. The original modulus of elasticity was reduced by 10% to match the experimental results.

The dynamic properties of ten typical multi-tiered temples are obtained using the finite element method in this paper. Three of these temples are tested using ambient vibration methods under wind-induced excitation to determine their true dynamic properties and validate their finite element models. An empirical formula is proposed to estimate the natural period of vibration of Nepalese temples based on the validated finite element model from the test. The seismic coefficient method is then used to evaluate seismic capacity. The response spectrum method is used in 3D dynamic analysis to achieve a more realistic result. The results show that the fundamental time period of masonry temples in Nepal is less than 0.6 s. Damping ratios range from 1% to 6%.[2].

One such article describes the findings of research endeavors to better recognise the engineering behaviour of ancient masonry historic sites at Thailand's Ayutthaya UNESCO World Heritage site. The bell-shaped and corn-shaped styles of Thai historic masonry monuments were studied. Both styles of monuments were analysed under their own weight using homogenised material properties in the first stage of modelling. The second stage included frequency analyses to determine their vibration properties and dynamic response ranges. Finally, two monuments' in situ ambient frequencies were measured and compared to analytical frequencies. Although the principal compressive stresses for both styles of monument were within safe limits, the safety factor against crushing around the corn-shaped monument's entrances was low[3].

The complexity of unreinforced masonry (URM) buildings makes model calibration a challenging task when updated models are required for predicting the response or behaviour of the structure. Research efforts have been made for updating models of URM structures. Aras et al. [4] determined the model properties of the Beylerbeyi Palace, which is an ancient three-storey masonry structure in Istanbul. They conducted ambient vibration tests and numerical investigations by using an FE model.

The original modulus of elasticity was reduced by 10% to match the experimental results. Jetson et al. used the modulus of elasticity as a calibration parameter for model updating because of the high uncertainty associated with it [5]. The preferred calibration parameter for URM and concrete structures is the modulus of elasticity due to its high uncertainty and variability [6]–[8]. Moreover, different procedures and methods have been used for updating the models of URM structures

4 Description of the Amrutesvara temple

The Archaeological Survey of India, Bangalore circle, Karnataka, provided the layout of the Amrutesvara temple (Figure 6). The layout consists of three components: the main temple, shrine for the subsidiary deity, and dining hall for devotees. The shrine and dining hall are on either side of the main temple. The main temple includes the garbhagriha, porch (a pillared hall provided for the devotees to stand and watch the worship rituals performed inside the garbhagriha), and open mandapa. The centreline and column layout of the temple are displayed in Figure 7. The dimensions of the temple in the plan are 14.85 m × 27.7 m × 10.5 m (height).

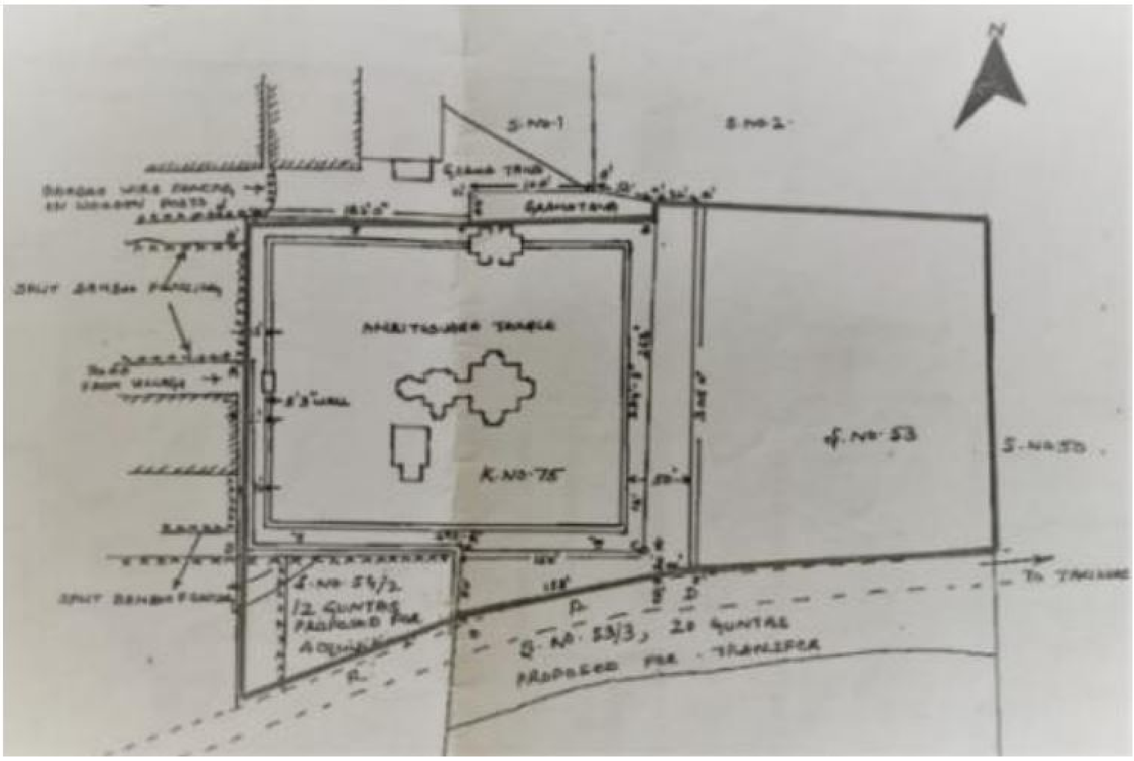


Figure 6: Layout of the Amrutesvara temple

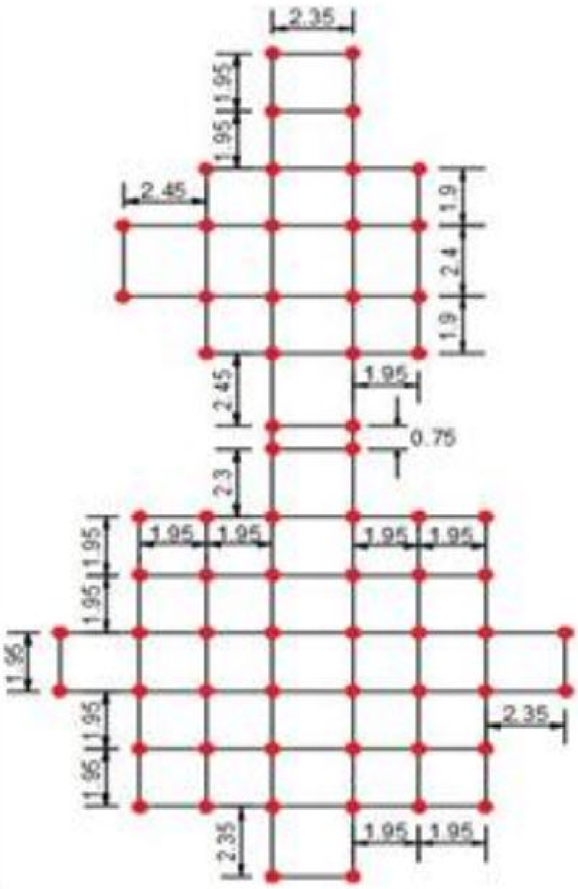


Figure 7: Centreline and column layout of the Amrutesvara temple

4.1 Properties of the materials used in the construction of the temple

The Amrutesvara temple is a dry-stone masonry structure that was constructed using soapstone. Three soapstone samples of size 15.24 cm × 15.24 cm were collected from a site near the Amrutesvara temple. Various tests were performed at the National Institute of Rock Mechanics (NIRM) to determine the material properties of these samples (Table 1). The L/D ratios maintained for the uniaxial compression test and Brazilian test were 2.0 and 0.5, respectively.

Loading data details have been provided in Table 1.

Table 1: Experimental results for the tested samples

Samplel.No	Density (kg/m^3)	Compressive strength (MPa)	Youngs modulus (MPa)	Poisson's ratio	Tensile strength (Mpa)
1	2813	45.16	32760	0.230	8.29
2	2796	51.47	36330	0.310	7.08
3	2787	50.01	31540	0.250	7.29
Avg	2798.6	48.88	33540	0.263	7.55

5 Free Vibration Test:

Two types of modal analysis exist: experimental modal analysis (EMA) and operational modal analysis. EMA is most commonly used modal analysis method. In EMA, an individual uses a device, such as a hammer, to stimulate a structure and then measures the response. The transfer function is then calculated. Specific modal parameter extraction algorithms are used to extract the dynamic properties of the structure. EMA, which is performed in four steps (Figure 8), has been used for design validation and FE analysis (FEA) verification [7].

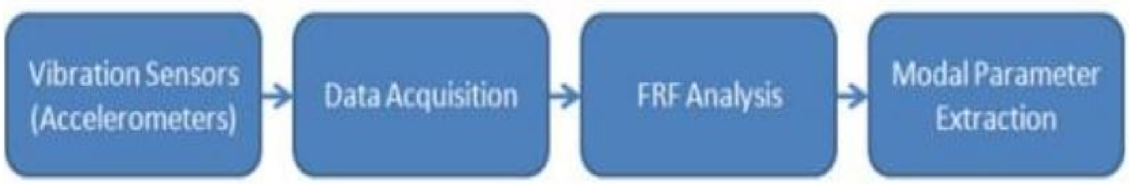


Figure 8: Four-step procedure for EMA

5.1 Vibration sensors (accelerometers)

Vibration sensors, also known as accelerometers, are appropriately placed on a structure (on various columns and beams) to record the response of the structure to a known excitation. The excitation must correctly excite the modes of the structural system, which indicate the modes of the structure. Excitation is provided by tapping the structure at a point.

5.2 Data acquisition

Specialised data acquisition (DAQ) hardware is required to obtain suitable vibration signals. The DAQ hardware used in this study was the Dynamic Signal Analyzer (DSA) provided by National Instruments, which accepts each channel with 24-bit high-resolution delta-sigma Analog to Digital Converter (ADCs). These DSA products have anti-aliasing filters that prevent aliasing and noise from affecting the measurement quality. They have suitable signal conditioning for power piezoelectric accelerometers [7]

5.3 Frequency response function analysis

The frequency response function (FRF) relates the stimulus and response for computing the transfer function of the building. The building's displacement magnitude and phase response are computed over a defined frequency range. Frequency response function displays the critical frequencies of the building, which are extra sensitive to excitation. The essential frequencies obtained from the analysis are the modes of the building under test.

5.4 Modal parameter extraction

The modal parameter extraction algorithms in LabVIEW were used to recognise the modal parameters from the FRF data. The algorithms include various methods of identifying the modal parameter, and each method is optimised for a specific test scenario. In the present experiment, the peak picking method was used.

5.5 Experimental setup and testing

Free vibration tests were conducted on the Amrutesvara temple to determine its dynamic response at various points. The excitation was related to the environmental loads and the manual shock for forced vibration. The vibration test was repeated to record the response at various points. The experiment was conducted using a four-channel DAQ with a piezoelectric sensor having a sensitivity of 10.52 mv/g. The sensor allowed acceleration responses to be recorded. The LabVIEW software was used for writing the algorithms to identify the modal parameter from the FRF data (Figure 9). The sampling rate maintained throughout the tests was 25 Hz because the frequency range of attention for most structures lies between 0 and 10 Hz [2] and because a sample size of 5000 was selected. In each test, the time histories and velocity were recorded and the signals converted to the digital form were saved in the hard disk of the DAQ computer. The experimental setup used at the Amrutesvara temple is displayed in Figure 10. Figure 11 displays the plot of acceleration versus frequency measured at different columns of the temple.

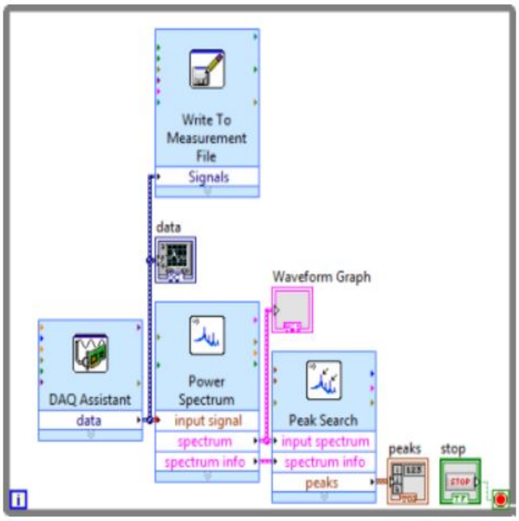


Figure 9: Modal parameter extraction algorithms

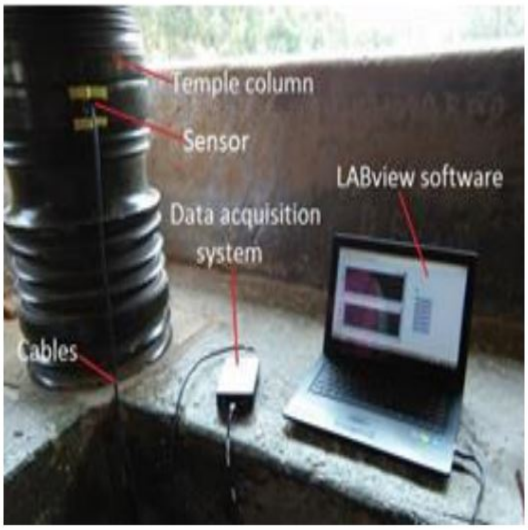


Figure 10: Experimental setup at the Amrutesvara temple

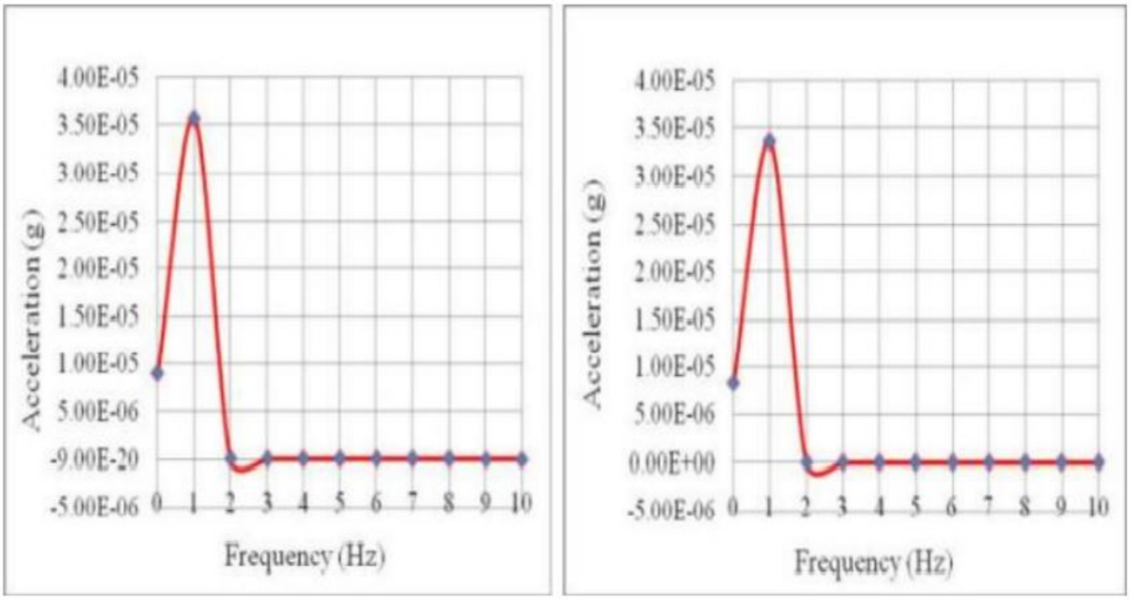


Figure 11: Acceleration versus frequency plot measured at different columns

6 Numerical Modelling

With the development of earthquake analysis procedures and modern computers, the response of an idealised structure to ground motion can be determined [2]. Because the geometry of historic buildings is complicated, the ANSYS program is used to achieve optimum performance, accuracy, and reliability in the prediction procedures [3]. The behaviour of the structure was studied using FEA. The FEA method was verified using the experimental values obtained from sensitive testing procedures, such as free vibration test. Measurements of various structural components were performed at the temple site by using a total station. These measurements were used to develop an AutoCAD 3D model. The properties of soapstone were obtained by conducting various tests at the NIRM. These properties were incorporated into the FE model. The building materials were assumed to be homogeneous, isotropic, and linearly elastic.

The FE model is exact replica of the existing structure. The tabled joint and cramped joint used in the column and beam connections as shown in figure 12 and 13 are also considered in the FE model. The beams are designed as the stepped beams displayed in figure 14. All the structural components that account for the mass and stiffness of the structure were considered for FE modelling. Constraints were applied to the joints;the base was considered to be rigid; and the domes were considered to rest on the beams. The pinnacle at the top, and balustrade of the open mandapa were not considered in the FEA. All the structural components that account for the mass and stiffness of the structure were considered for FE modelling. Constraints were applied to the joints; the base was considered to be rigid; and the domes were considered to rest on the beams. The pinnacle at the top, and balustrade of the open mandapa were not considered in the FEA.

Figure 12: Tabled joint in the columns

Figure 13: Cramped joint used in the beam connections

Figure 14: Stepped beams

Initially the temple model was designed using AutoCAD 3D based on the dimensions and the geometrical shapes of the structural members as seen in figure 15. The finite element analysis was carried out using ANSYS Software. All the components of the temple are meshed with the combination of hexahedral and tetrahedral elements of 1st order. Node to node connectivity has been ensured at all locations. The aforementioned elements were connected so that the FE mesh complied with the required constraints. A connected mesh is the core of FE problems. The solid bottom elements were constrained in all directions. Also the critical areas have been modelled with finer mesh to ensure the accuracy of the mesh. The total number of elements in the model is 40, 39,782. Total number of nodes is 19, 25,289.

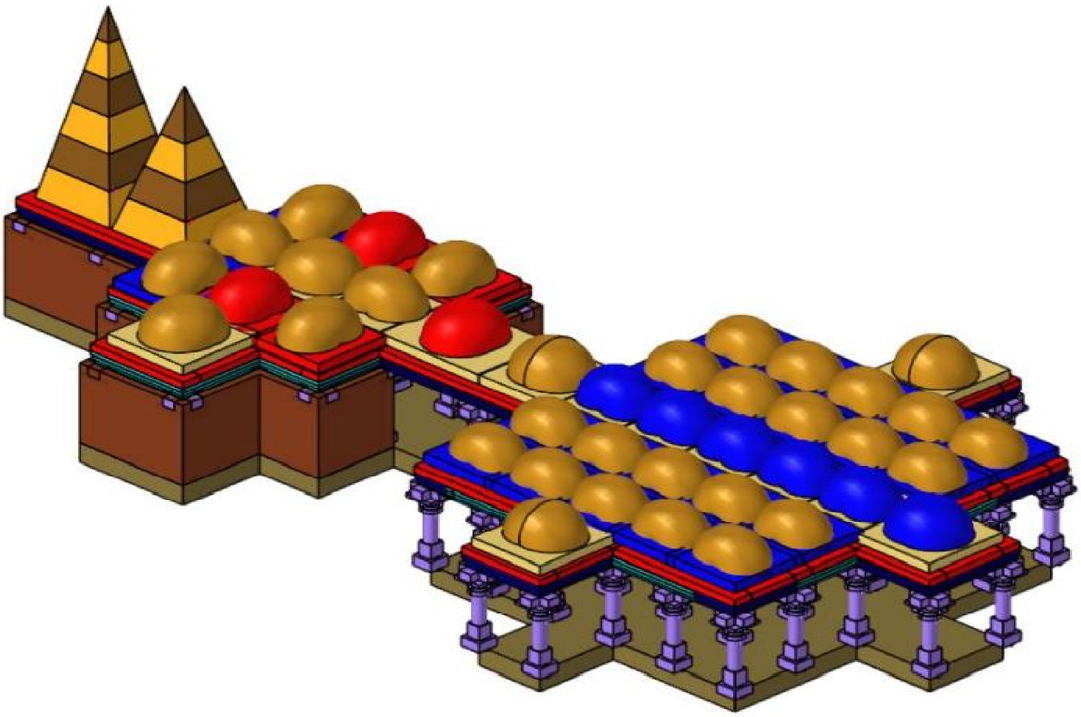


Figure 15: Autocad 3D drawing of the entire temple

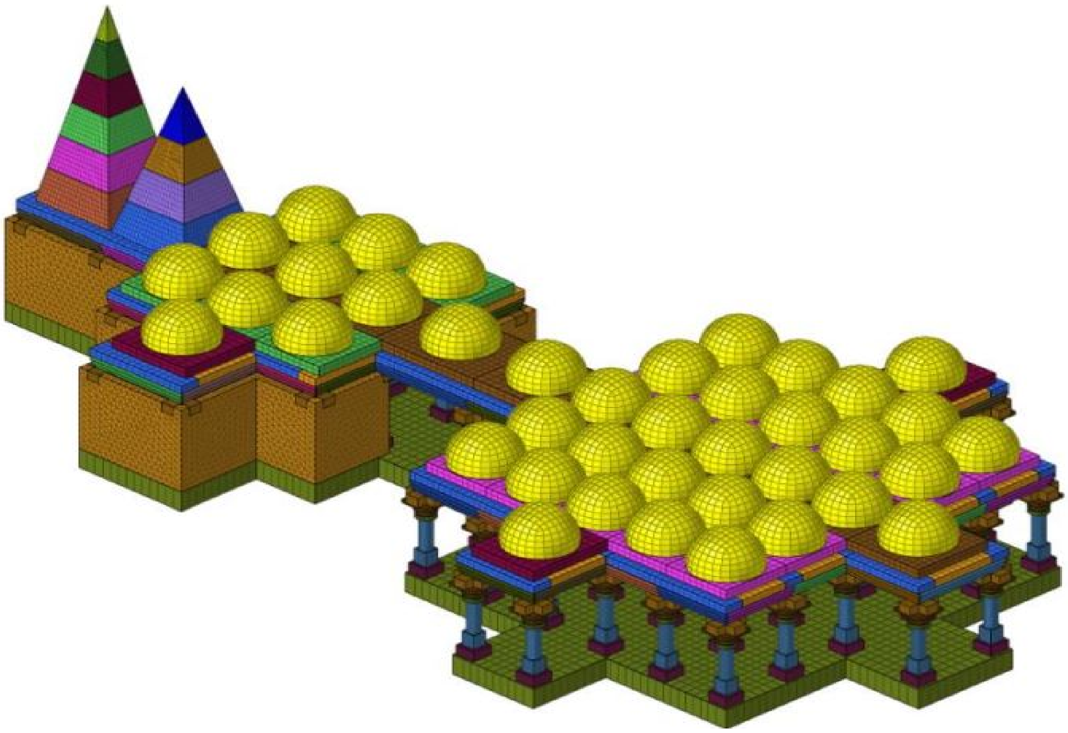


Figure 16: FE model of the entire temple

7 Results and Discussions

The experimental and numerical natural frequencies of the Amrutesvara temple are listed in Table 2. A significant difference was observed between the experimental and FEA results. The reasons for this difference were analysed. The temple structure contains visible cracks and also some cramp used in the beam connections are missing displayed in figure 17 and 18. Considering the visual damage, the E value measured in the lab on a small specimen is less than the E value of the temple structure because of the damage and cracking of the structural elements and also because of the size effect in the quasi-brittle structure. The difference in measured and calculated frequencies of the structures can also be seen in the analysis of various ancient structures [1][4][5][6][7]. The mode shapes obtained from FEA are displayed in Figures 19–21. As expected, the fundamental modes is associated with torsion because of the irregularity in mass and stiffness.

Figure 17: Missing clamps at beam connections

Figure 18: Visible cracon the structural members

Table 2: Comparison of the experimental and numerical fundamental frequencies obtained for the Amrutesvara temple

Model	Numerical Model Frequency (Hz)	Ambient vibration test frequency (Hz)
Amrutesvara Temple	1.46	1.1

Table 3: Dynamic properties of the temple obtained from the 3D FE model

Model	Frequency (Hz)	Mode of Vibration
1	1.46	X-Translation
2	1.88	Torsion
3	1.9	Y-Translation

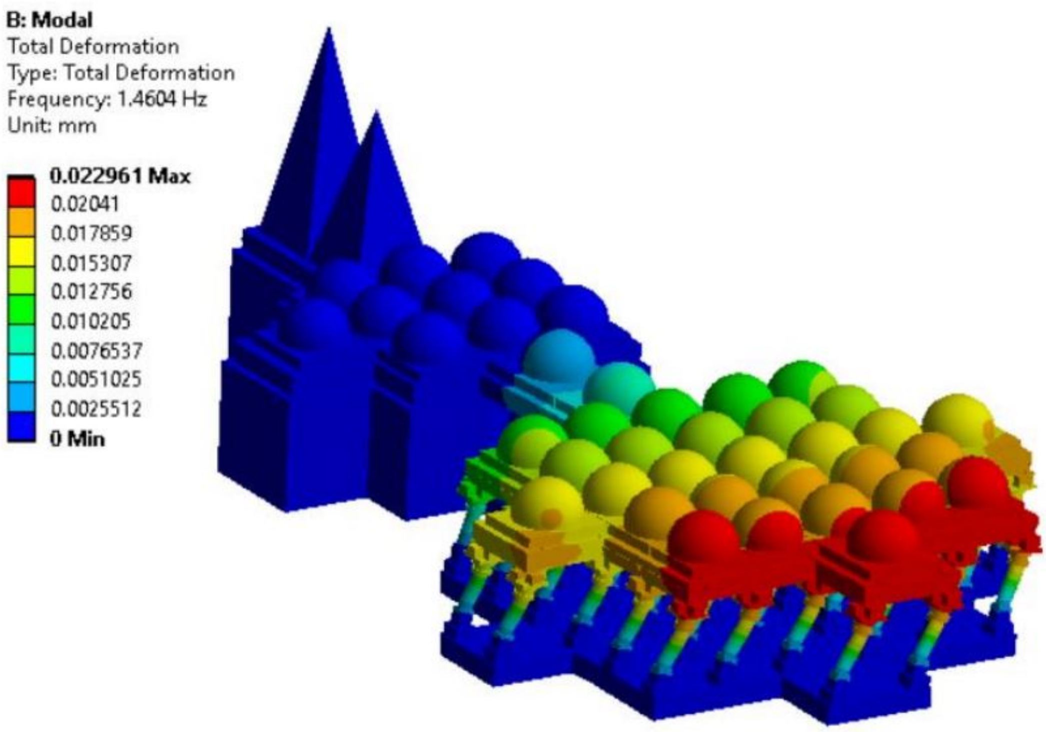


Figure 19: Shape of the first mode

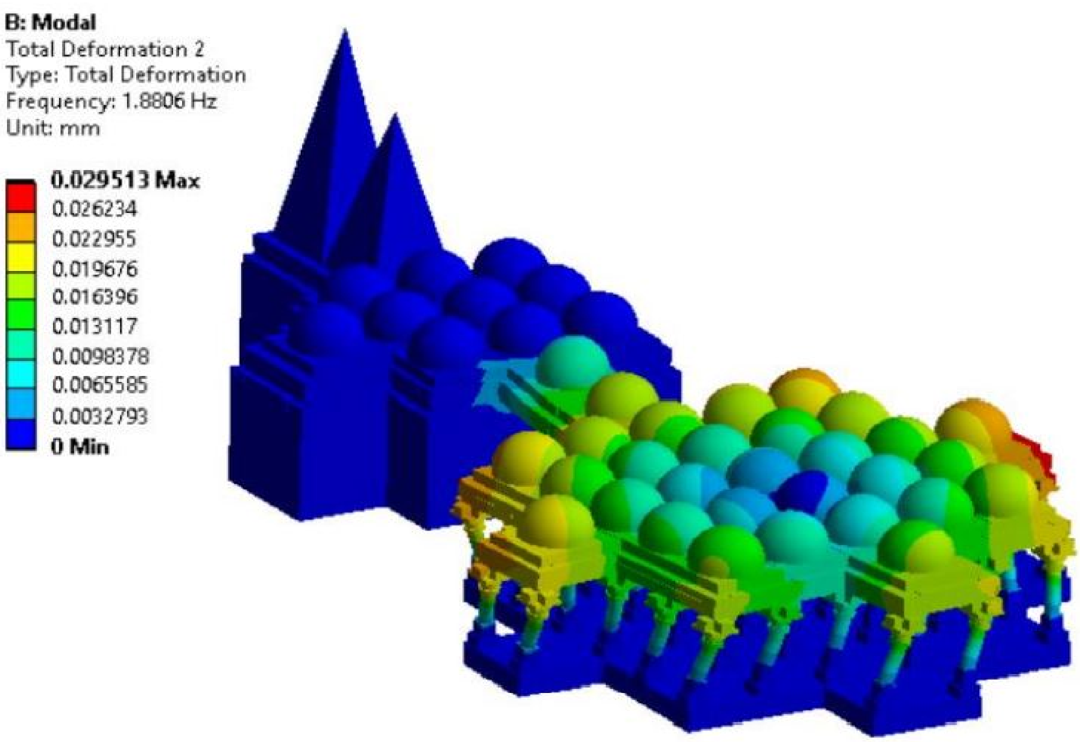


Figure 20: Shape of the second mode

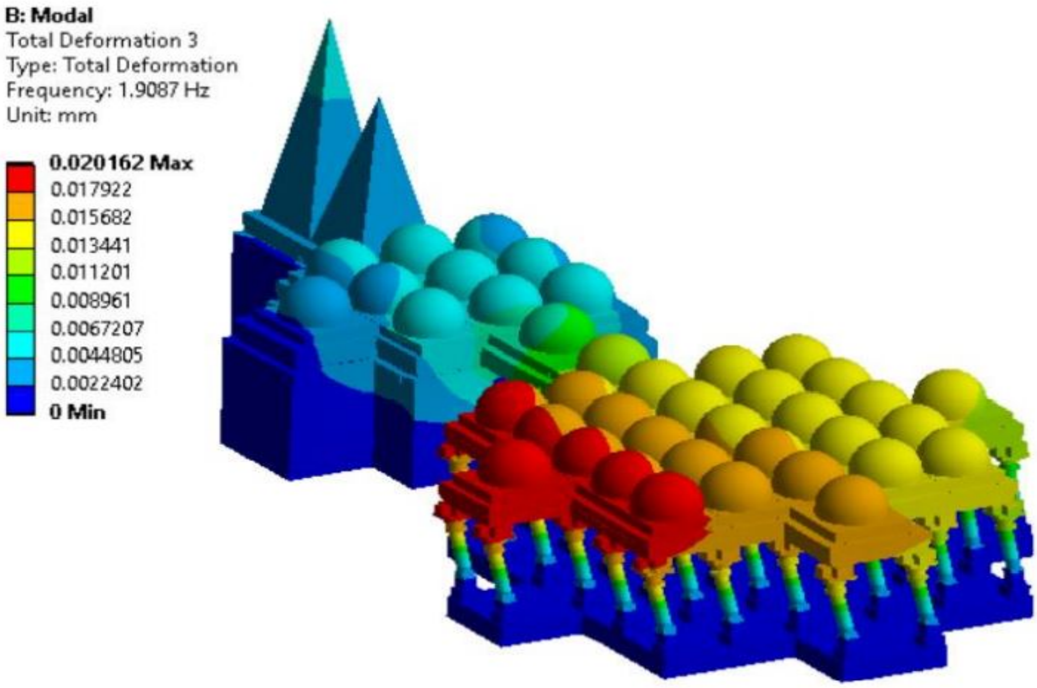


Figure 21: Shape of the third mode

8 Conclusions

The objective of this research was to perform seismic analysis of the Amrutesvara temple, which is a heritage stone masonry structure in south India. The research work was planned in various stages. This paper presents the results obtained in the experimental and numerical analysis of the temple. The natural frequency of the temple measured in the free vibration tests was 1.1 Hz. The calculated natural frequency obtained through numerical modelling was 1.46 Hz, which indicates that the structure is stiff. The difference in experimental and numerical result is because of the visible cracks and missing clamps at joints. Therefore, to reduce the stiffness of the structure and to replicate the present condition of the temple, the Young's modulus need to be updated based on damage assessment analysis.

Torsion was observed as the second mode of vibration in the analysis due to the irregularity in the temple plan. The modulus of elasticity and material properties used in this investigation were derived from experiments. Therefore, an accurate geometric model of the temple can be constructed using solid elements with the aforementioned material properties. Such a 3D FE model can be used for further investigation of the temple. Moreover, the methodology used in this study can be adopted for designing 3D FE models for other temples with Hoysala architecture to study their seismic characteristics and to suggest suitable retrofitting measures for protecting them.

Acknowledgements

The research was supported by the Vision Group of Science and Technology (VGST). The authors acknowledge the Archaeology Department of Karnataka for providing the structural details of the Amrutesvara temple; the National Institute of Rock Mechanics, Kolar, for testing the stone samples; as well as the Central Power Research Institute and K. S. School of Engineering and Management for their encouragement. The authors also thank the reviewers for their helpful comments and suggestions.

References

[1] Ronald J.A, Menon A, Prasad A.M, Menon D, Magenes G. Modeling and Analysis of South Indian Temple Structures Under Earthquake Loading. Indian Academy of Science., May 2018

[2] Jaishi B, Ren W, Zong Z.H, Maskey P.N. Dynamic, and Seismic Performance of Old Multi-tiered Temples in Nepal. Eng.Struct 2003; 25(14), 1827-1839

[3] Wethyavivorn B, Surit S, Apichatyakul C, Lerdsivanon N. Model Verification of Thai Historic Masonry Monuments. J. Perform. Constr. Facil., 2016, 30(1): 04014188

[4] Aras, F., Krstevska, L., Altay, G., and Tashkov, L. (2011). "Experimental and numerical modal analyses of a historical masonry palace." Construction and Building Materials, 25(1), 81-91.

[5] Maeda T, Sugiura Y, Hirai T. FEM modeling of the towers in Bayon temple in Cambodia based on micro-tremor measurements. Adv. Eng. Softw. 39(4): 346–355, 2008.

[6] Ventura C. E, Brincker. R., Dascotte E, Andersen, P. FEM updating of the heritage court building structure. Proc., International Modal Analysis Conference (IMAC-XIX), 2011, 1, 324-330

[7] Wood, R. L., Mohammadi, M. E., Barbosa, A. R., Abdulrahman, L., Soti, R., Kawan, C. K., Shakya, M., and Olsen, M. J. (2017b)."Damage Assessment and Modeling of the Five-Tiered Pagoda-Style Nyatapola Temple." Earthquake Spectra

[8] .Abdulrahiman L.M. Finite Element Modelling and Updating of a Five-Tiered Pagoda Style Temple. Partial fulfillment of the requirements for the Degree of Master of Science in civil engineering. University of Nebraska-Lincoln, May 2001.

[9] Shakya M. Modal Analysis Using Ambient Vibration Measurement and Damage Identification of three-tiered Radhakrishna temple. Partial fulfillment of the requirements for the Degree of Master of Engineering in Earthquake. Khwopa Engineering College, Bhaktapur, Purbanchal University, Nepal, October 2010

[10] Modal Analysis in NI LabVIEW – user's manual. National Instruments, March 2016

[11] . LabVIEW User's Manual, National Instruments, April 2003.

[12] Indian Standard. Methods of Test for Determination of Strength Properties of Natural Building Stones. IS: 1121 (Part-I) 1974.

[13] Indian Standard. Method for Determination of Modulus of Elasticity and Poisson's Ratio of Rock Materials in Uniaxial Compression. IS: 9221-1979

EVALUATION OF SEISMIC CAPACITY OF BUILDING STRUCTURES RESTING ON SLOPING GROUND BY PUSHOVER ANALYSIS

Ms. Zaiba Fathima [1] , Mr. Naveean M P[2] , Dr.Vijayalakshmi Akella [3]

1. P G Student: Department of Civil Engineering, K S School of Engineering & Management, Bangalore, India

2. Assistant Professor : Department of Civil Engineering, K S School of Engineering & Management, Bangalore, India

3. Professor and Head : Department of Civil Engineering, K S School of Engineering & Management, Bangalore, India

zabfathimaks@gmail.com

Abstract

In building structure resting on sloping ground the Centre of Mass and the Centre of Stiffness are not congruent, putting them at increased risk for shear and torsion because of their irregularity and asymmetry. The building models under consideration are subject to seismic forces. The models of the buildings under consideration are analysed by nonlinear static pushover analysis. The analysis's findings regarding base shear and displacement at the performance point are then contrasted among the building configurations taken into consideration. In this study G+10 storey building is subjected to nonlinear analysis and the behaviour of the structure is obtained through pushover curve and performance point of the structure. In these analysis four models studied is Fixed Base system resting on $15_{\circ}$ and $27_{\circ}$ sloping ground and Flexible Base system resting on $15_{\circ}$ and $27_{\circ}$ sloping ground. The analysis is done using ETABS software. Hinges are defined in beam-column joints under auto hinge property using FEMA 440 and ASCE 7- 10 as reference. Nonlinear push loading is applied to the structure. Based on the results, the base shear at performance point is higher in fixed base system compare to flexible base system, Base shear increases with increase in slope of the ground where as displacement decreases. Formation of hinges beyond CP (collapse prevention) is more as slope of the ground increases in fixed and flexible base system

Keywords:
Sloping ground, pushover analysis, Base shear

1 INTRODUCTION

The most dangerous events are earthquakes because of their unpredictable nature and immense destructive force. People are not killed by earthquakes directly, instead massive losses in human life and property result from the demolition of structures. Strong earthquakes can force building structures to collapse, which directly costs lives. Over the past few decades, numerous studies have been conducted across the world to determine what causes various types of structures to fail when subjected to intense seismic activity. In a mountainous environment, the lack of flat land forces development to take place on slopes. When exposed to earthquake ground vibrations, hill structures built in masonry with mud or cement mortar and not in compliance with seismic code regulations have proven dangerous and caused loss of life and property. Pushover analysis has proven to be the better solution to analyse such structure to check their capacity against seismic force.

2 Literature Review

Twenty four RC structures in 3 distinct configurations, a 3-D study with torsional impact have been performed on sloping land, set back structures area unit thought-about to be additional suited. Once subjected to unstable excitation, Step-back structures might perform worse than alternative building than Step-back set back buildings. The intense left column at ground level, that is noted to be the worst affected, ought to receive additional care in terms of style and details. Step-back buildings develop torsional moments at the next rate than Step-back set back structures.[1].

The study was carried out on 3 totally different configurations that's Step-back Set-back, Setback, Step-back G+3 storied structure on sloping and flat land area unit examined. From this investigation, the subsequent conclusions were created. Step-back induces high base shear. Constructing building on level land with quantity amount of blow. Step-back building's high floor displacement is fairly important compared to a structure that's set back and rests on a slope ground. On slopes, step back-set back construction could also be most well-liked.[2] .

Analysis of the building by exploitation structural analysis software ETAB, to check the impact of varied height of the column in bottom structure at totally different positions throughout the earthquake. The results are compared with the results of the building with and while not level ground (step back) for G + 12 storeys RCC building and therefore the ground slope for twenty, thirty & forty degree is thought-about. Fundamental quantity and high structure displacement will increase with increase in variety of story level. Step-back buildings turn out higher worth of your time amount, high structure displacement and base shear compare to Step-back Set-back building frames. Base shear, structure drift decreases with increase with increase in slope. [3]

Pushover analysis was carried out to calculate the force-displacement relationship, or capability curve. RC frame structure's performance was assessed employing a Utilizing ETABS 9.7.1, nonlinear static pushover analysis has been administrated on 5, 10, and 15 storeys of RC clean frame constructions were examined. Likewise, the fundamental force was compared and therefore the displacement of a 5,10 and 15 story RC clean frame building. As at performance purpose, hinges were in L S-C P vary, overall performance of building is claimed to be Life safety to Collapse bar level. Out of eighty assigned hinges fifty four were in A-B stage, 12, 10, and 4 hinges area unit in IO-LS and LS-CP stages severally. In fifteen story frame structure pushover analysis was together with twelve steps. The RC clean frame that is analysed for the static nonlinear cases will carry higher base force and at lower displacement it fails. [4]

Case was done study on "April 2015 Gorkha Earthquake" was tragic in terms of the death toll, loss of history, structures, and feelings. Additionally, a house located at Purano Naikap-13 in Kathmandu, Kingdom of Nepal, was severely damaged. The building was assessed to be in good condition; the Life Safety threshold was not exceeded during earthquakes based on style. Following the retrofitted columns, as mentioned in the preceding sections, the building's foundation story drift and deflection significantly improved, according to structural analysis carried out using the ETABS computer code. Inter-story drift was significantly decreased in both the X and Y directions. Reduced high storey deflection previously, the ground level was utilized for parking, with thin walls enclosing the building's exterior. With 230 metric linear unit brick walls and other comparable barriers, upper stories were utilized as residential floors. The difference in stiffness between the brick walls on the ground level and upper stories may have been the main factor in the building's damage during the earthquake. [5]

3 Modelling

Performance evaluation of RCC framed structure on sloping ground with pushover analysis. A G+10 storey reinforced concrete building has been modelled according to the plan dimensions, storey number and desired storey height and angle $15_{\circ}$ and $27_{\circ}$ on fixed and flexible base as shwon in Table 1.

3.1 Building Description

Table 1: Model Description

Parameters	
Plan Size	21*15 m
Bottom storey height	1.75 m
Storey height	3.5 m
Angle of Slope	150 and 270
Base Type	M30 concrete and Fe500 steel Masonry specific weight $20KN/m^3$
Materials for beam	M35 concrete and Fe500 steel
Materials for column	0.2 m
Materials for slab	M25 concrete and Fe500 steel
Column Size	0.5 X 0.6 m
Beam Size	0.3 X 0.45 m
Live Load	3 kN/m^2
Slab thickness	0.15 m
Wall Load	11 kN/m^2

3.2 Plan of models :

The model developed for the above said building specifications has been presented fromFigure 1- Figure 5

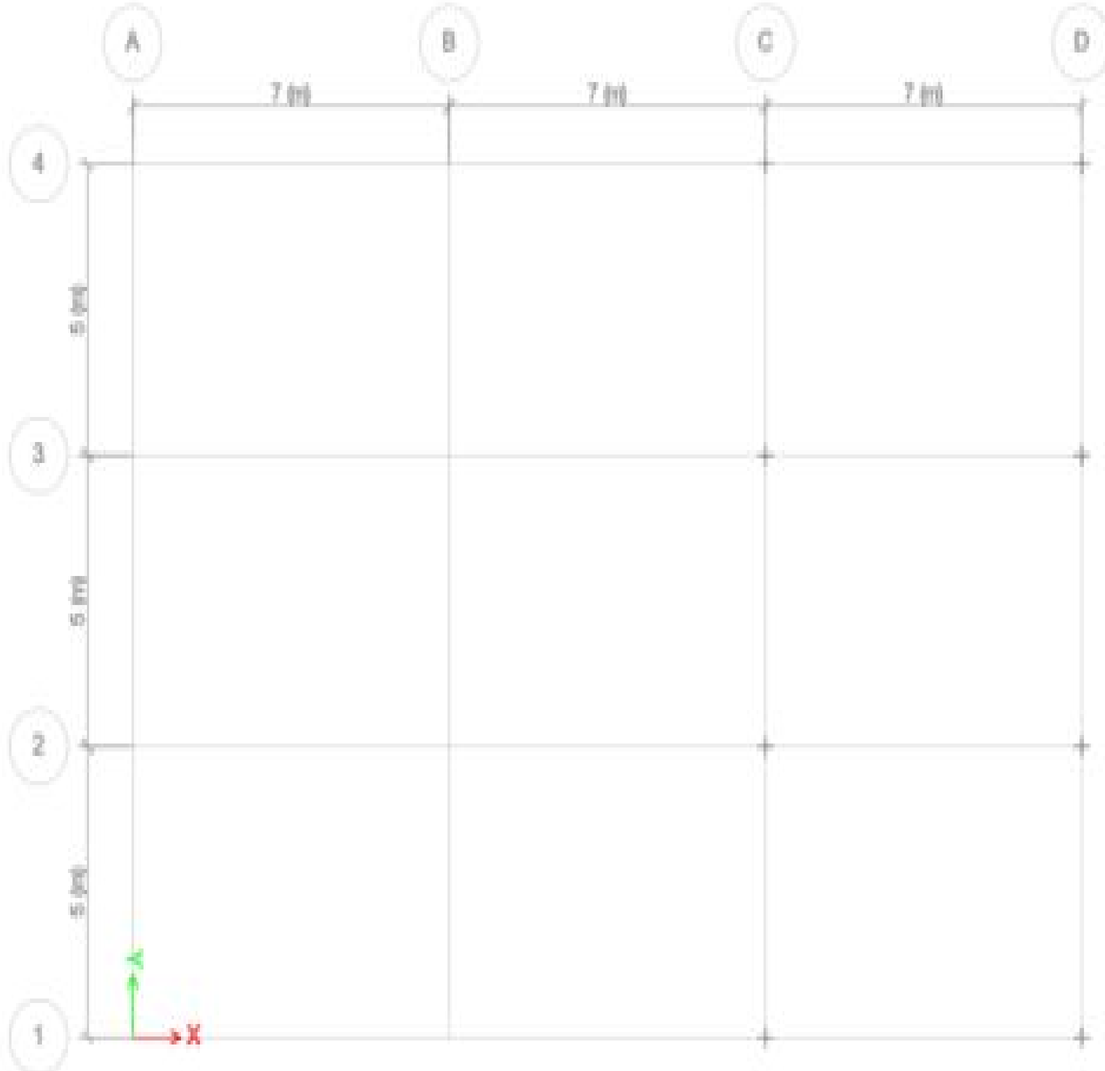


Figure 1: Floor plan of model

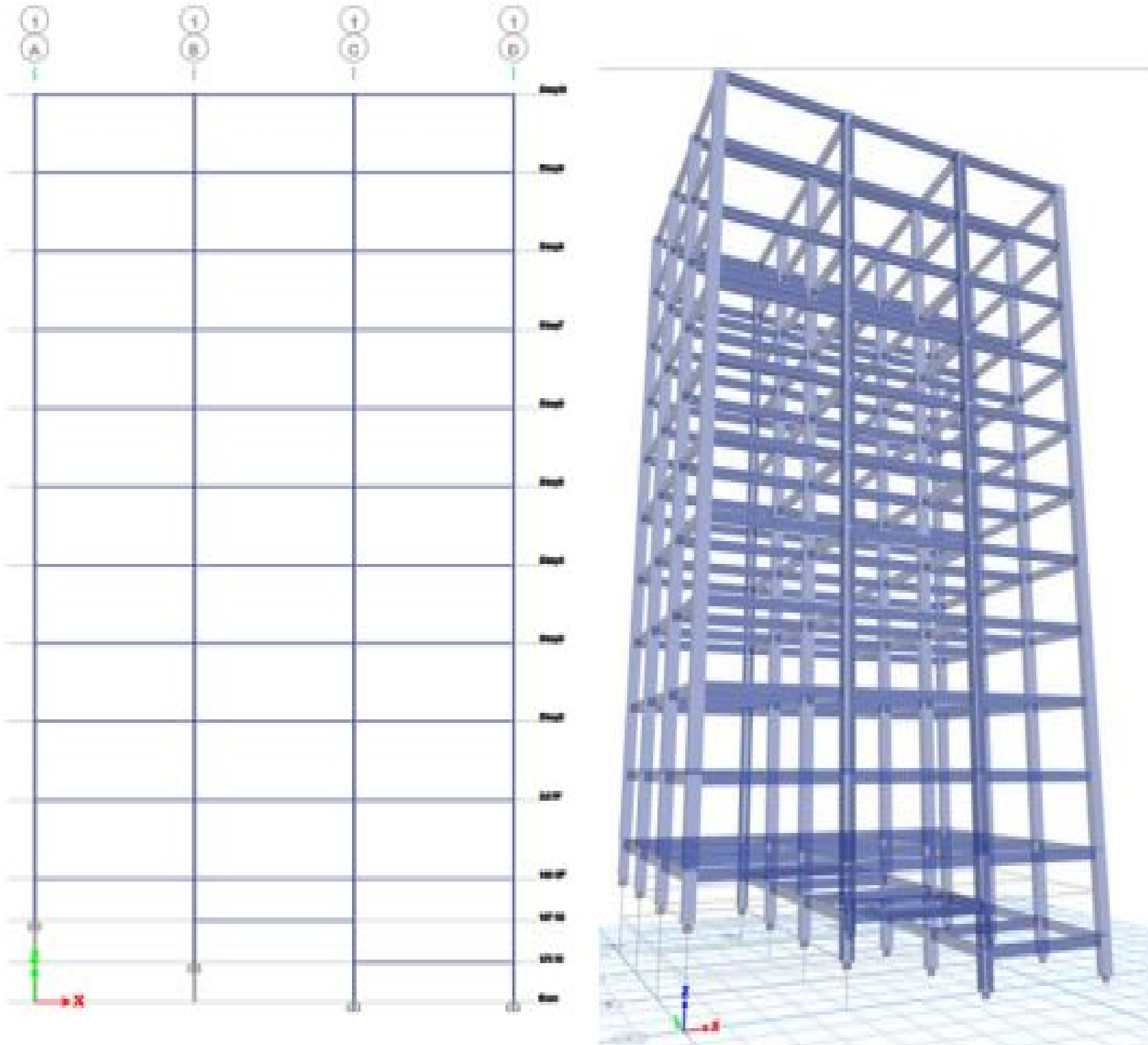


Figure 2: Front elevation and 3D view of 15^0 model with fixed base

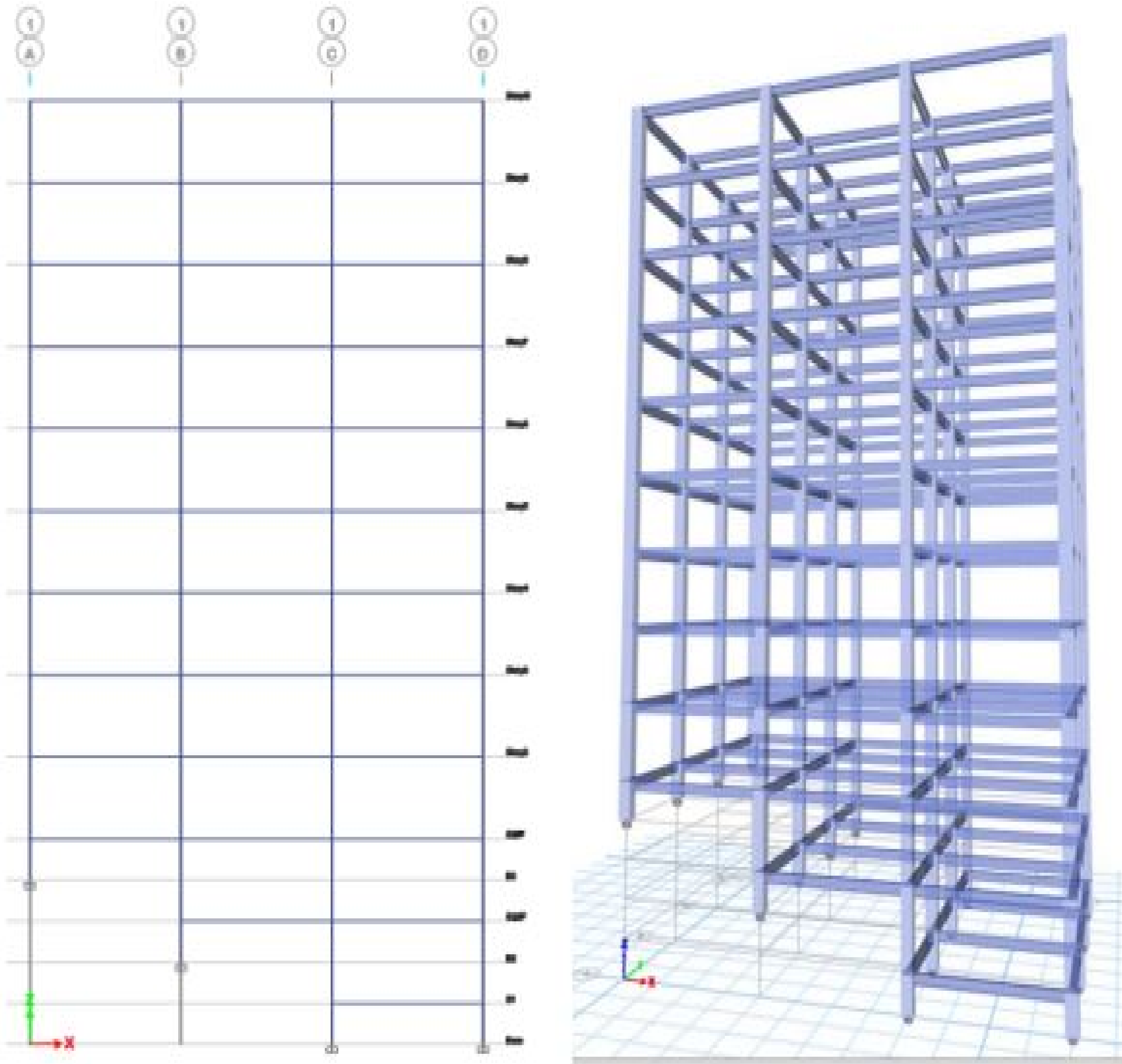


Figure 3: Front elevation and 3D view of 27^0 model with fixed base

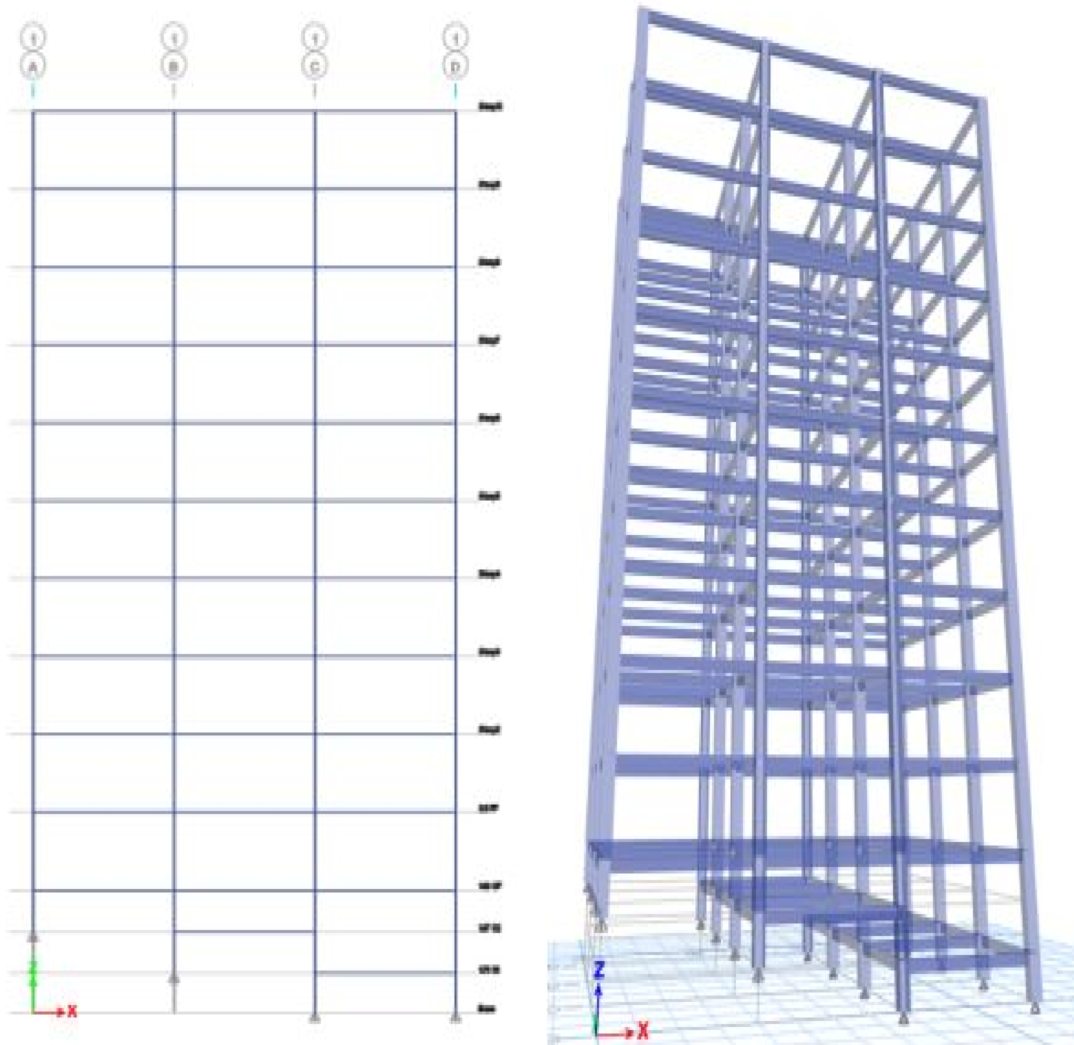


Figure 4: Front elevation and 3D view of 15^0 model with flexible base

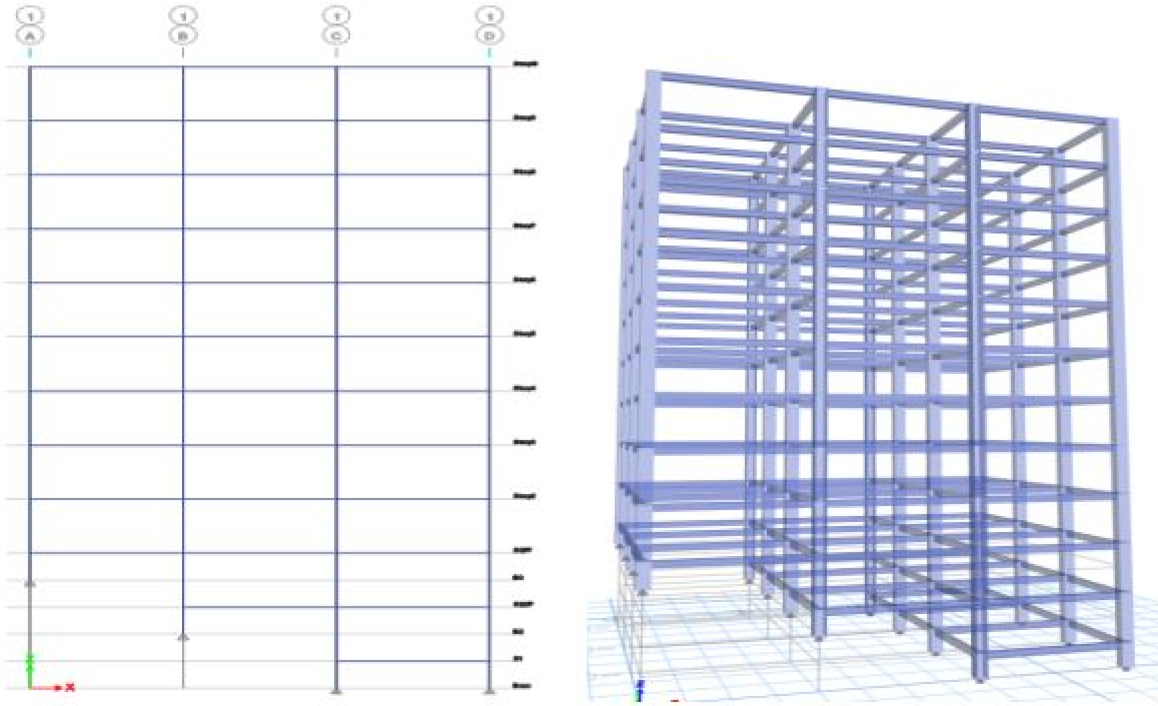


Figure 5: Front elevation and 3D view of 27^0 model with flexible base

4 Results and Discussions

4.1 Fixed Base system

4.1.1 15^0 Slope

(Figure 6)Table shows the hinge state for Base Shear and Monitored Displacement of the fixed base building resting on 15$_\circ$ sloping ground. The total hinges formed are 1904.The hinges are formed when the displacement has reached 296mm at LS state and the hinges are formed beyond CP when the displacement has reached 544.45mm. Figure shows the final hinge formed beyond CP state, located at extreme right first floor columns of the building with fixed base which is resting on 15$_\circ$ sloping ground.

TABLE: Base Shear vs Monitored Displacement

Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	1904	0	0	0	0	1904	0	0	0	1904
1	61.142	964.8	1896	8	0	0	0	1904	0	0	0	1904
2	93.858	1291.1	1712	192	0	0	0	1904	0	0	0	1904
3	134.45	1481.6	1600	304	0	0	0	1904	0	0	0	1904
4	296	1817.5	1488	416	0	0	0	1896	8	0	0	1904
5	458.5	2029.1	1388	516	0	0	0	1740	164	0	0	1904
6	544.45	2096.9	1344	560	0	0	0	1680	220	0	4	1904
7	571.96	2111.7	1344	560	0	0	0	1632	268	0	4	1904
8	572.02	2111.6	1344	560	0	0	0	1626	274	0	4	1904
9	574.87	2113.2	1340	564	0	0	0	1624	276	0	4	1904
10	574.88	2113.1	1340	564	0	0	0	1624	276	0	4	1904
11	574.94	2113.4	1340	564	0	0	0	1624	276	0	4	1904
12	575.3	2113.7	1340	564	0	0	0	1622	278	0	4	1904
13	575.3	2113.5	1340	564	0	0	0	1622	278	0	4	1904
14	575.36	2113.8	1340	564	0	0	0	1622	278	0	4	1904
15	575.84	2114.2	1340	564	0	0	0	1622	278	0	4	1904
16	575.87	2114.1	1340	564	0	0	0	1622	278	0	4	1904
17	575.89	2114.1	1340	564	0	0	0	1622	278	0	4	1904
18	575.9	2114.1	1340	564	0	0	0	1622	278	0	4	1904

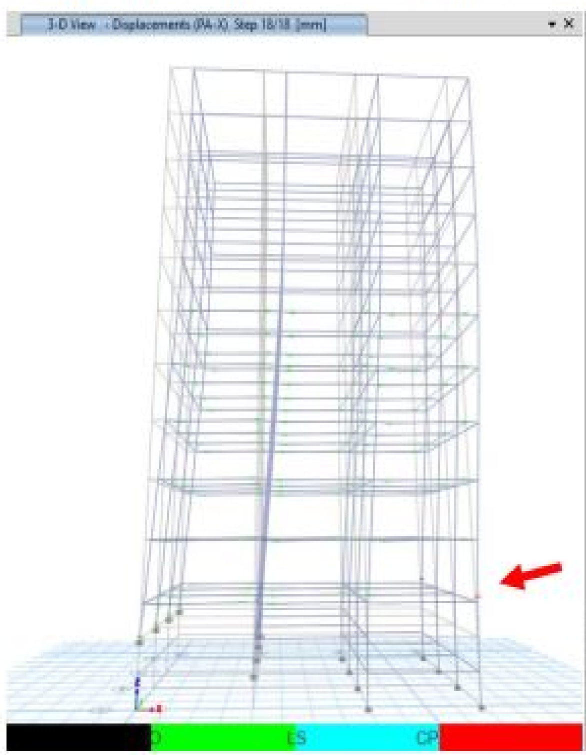


Figure 6: Capacity curve Coordinates for Base Shear and Monitered Displacement with Final Hinge formations

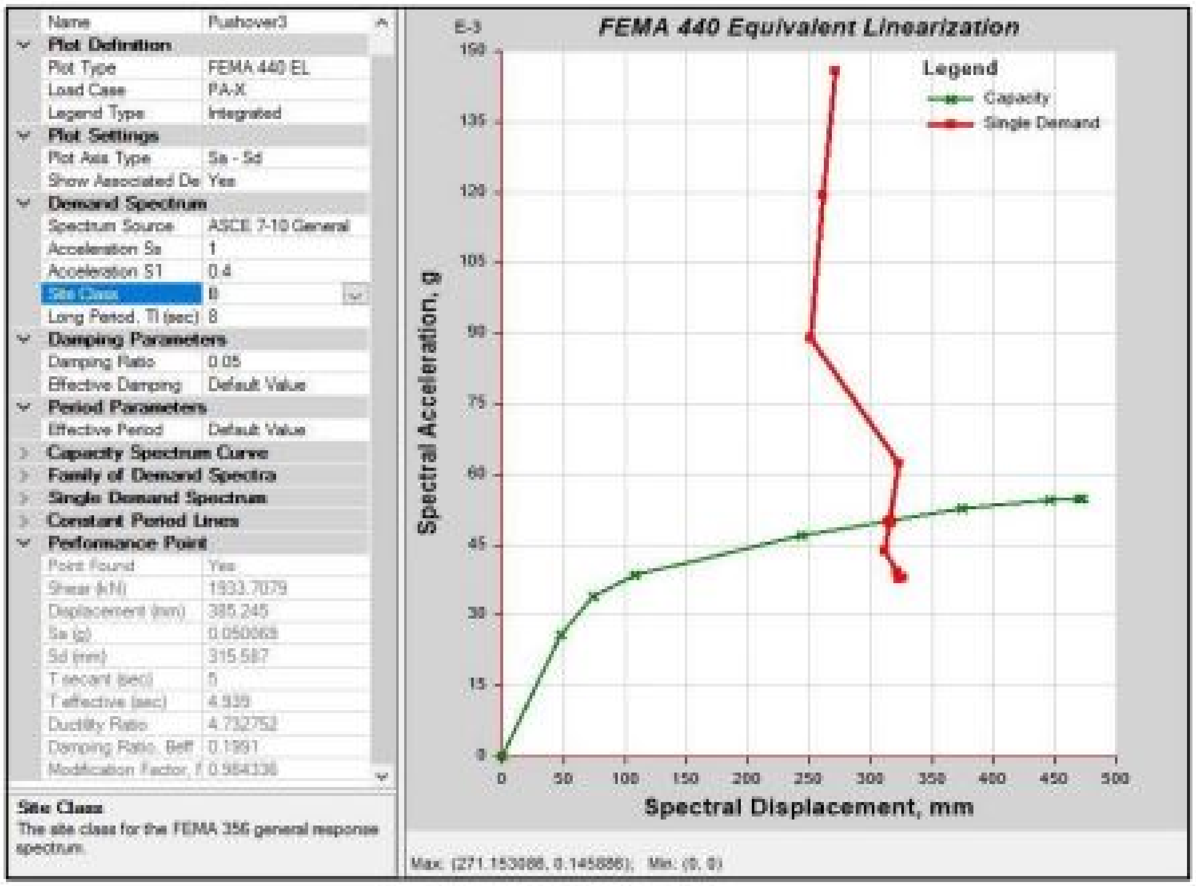


Figure 7: Pushover Curve-FEMA 440 Equivalent Linearization

(Figure 7) shows the variation between spectral acceleration (Sa) and displacement (Sd).It is observed that the curve is nonlinear in nature where the curves represent capacity and demand. The performance point is found at 0.050g spectral acceleration (Sa) and 315.587mm and spectral displacement (Sd) where the base shear is 1933.7079 KN and displacement 385.245 mm

4.1.2 27^0 Slope

(Figure 8)Table shows the hinge state of the fixed base building resting on 27◦ sloping ground. The total hinges formed are 1788.The hinges are formed when the displacement has reached 280 mm at LS state. The hinges are formed beyond CP when the displacement has reached 620.36 mm.: Figure shows the final hinge formed beyond CP state, located extreme right first floor columns and base of second step columns of the building with fixed base which is resting on 27◦ sloping ground.

TABLE: Base Shear vs Monitored Displacement

Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	1788	0	0	0	0	1788	0	0	0	1788
1	48.78	928.59	1784	4	0	0	0	1788	0	0	0	1788
2	74.979	1353.4	1688	100	0	0	0	1788	0	0	0	1788
3	106.96	1587.4	1572	216	0	0	0	1788	0	0	0	1788
4	280	2072.9	1420	368	0	0	0	1780	8	0	0	1788
5	438.51	2338.2	1332	456	0	0	0	1624	164	0	0	1788
6	576.77	2455.5	1252	536	0	0	0	1556	232	0	0	1788
7	620.36	2479.2	1232	556	0	0	0	1522	264	0	2	1788
8	625.93	2481	1228	560	0	0	0	1518	268	0	2	1788
9	706.43	2495	1228	560	0	0	0	1506	280	0	2	1788
10	912.71	2534.5	1216	572	0	0	0	1404	380	0	4	1788
11	952.96	2543.7	1216	564	8	0	0	1372	408	0	8	1788
12	963.03	2544.8	1216	560	12	0	0	1368	404	8	8	1788
13	973.09	2544.8	1216	556	16	0	0	1364	404	12	8	1788
14	983.15	2543.9	1216	532	40	0	0	1364	400	16	8	1788
15	995.73	2535.6	1216	504	68	0	0	1364	372	44	8	1788
16	1028.4	2472.7	1216	448	124	0	0	1364	310	106	8	1788
17	1073.7	2327.6	1216	422	142	8	0	1360	272	148	8	1788

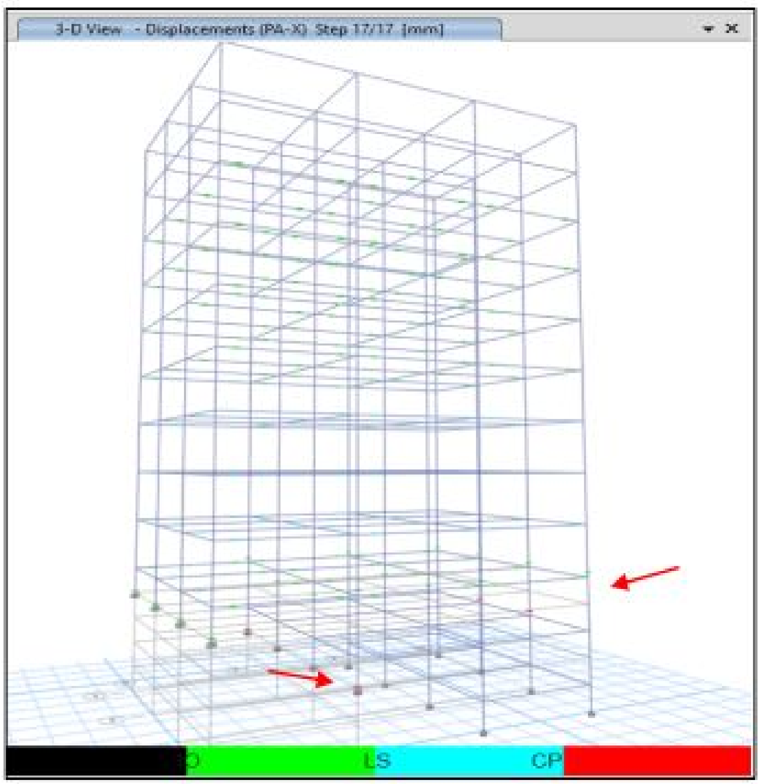


Figure 8: Capacity curve Coordinates for Base Shear and Monitered Displacement with Final Hinge formations

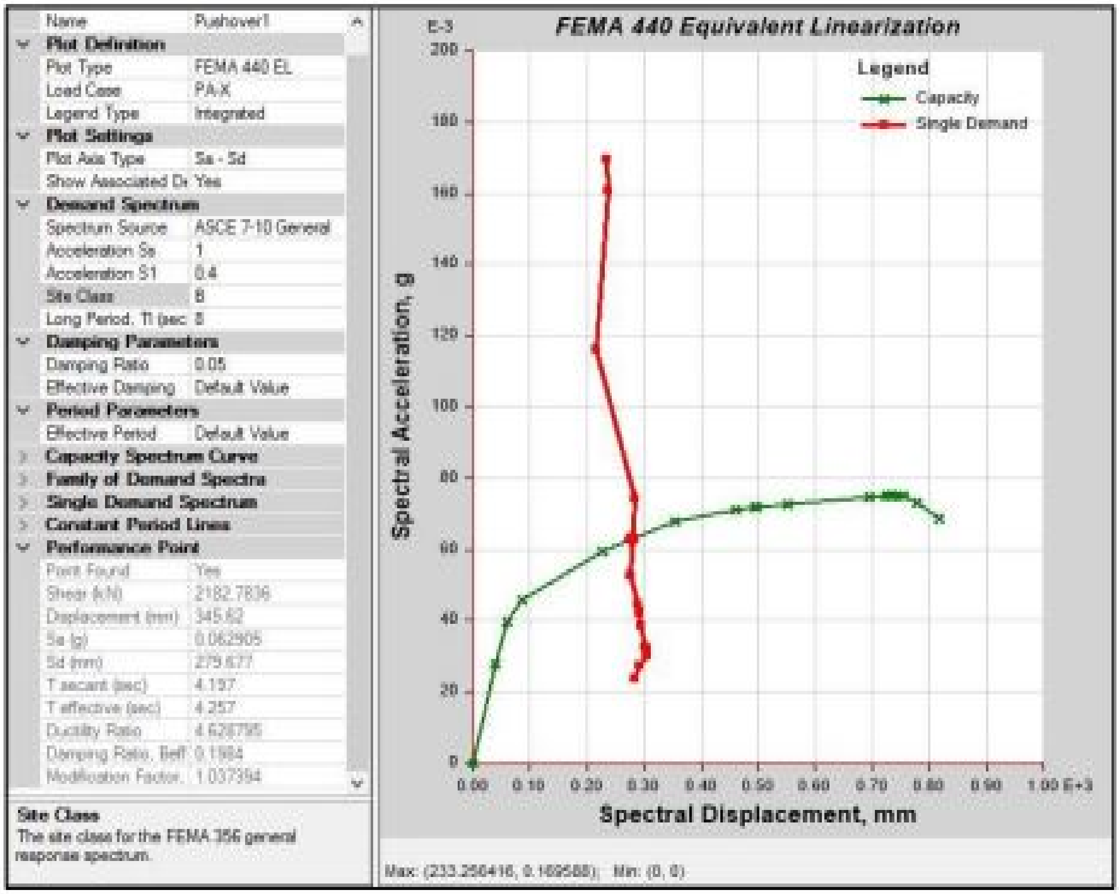


Figure 9: Pushover Curve-FEMA 440 Equivalent Linearization

Figure 9 shows the variation between spectral acceleration (Sa) and displacement (Sd).It is observed that the curve is nonlinear in nature where the green curve represents capacity and red curve represents demand. Performance point is found at 0.062905g spectral acceleration (Sa) and 279.677 mm and spectral displacement (Sd) where the base shear is 2182.7836 KN and displacement 345.62 mm

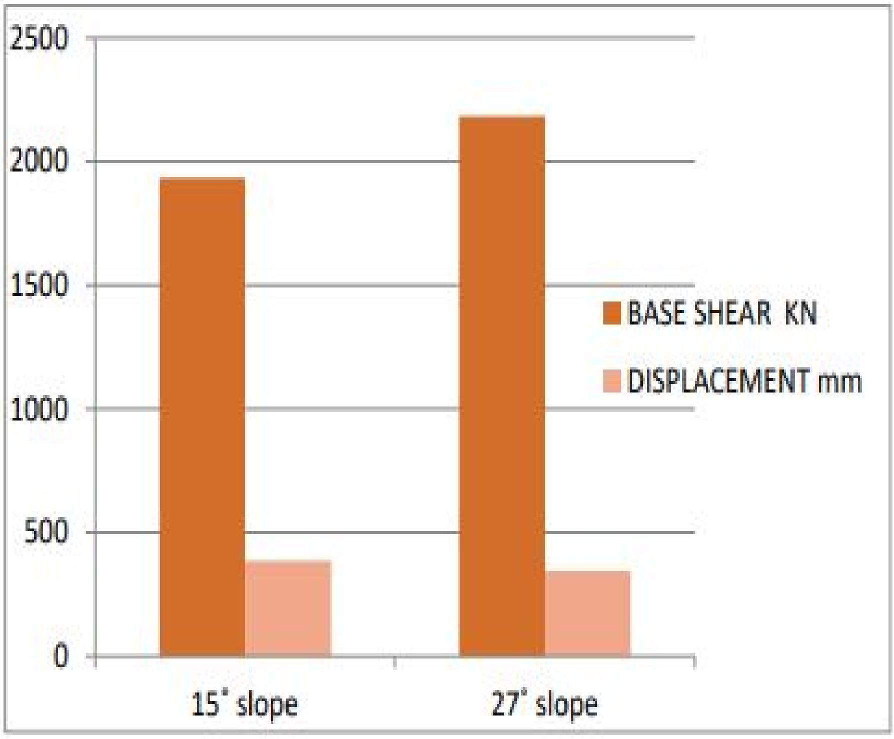


Figure 10: Base Shear and Displacement comparison of Fixed Base System

The Figure 10shows the variation between base shear and displacement v/s angle of slope of the fixed base building structure resting on 15◦ and 27◦.The base shear of 15◦ and 27◦ are 1933.70 KN and 2182.78 KN and the displacements are 385.245mm and 345.62mm respectively

4.2 Flexible base system:

4.2.1 15^0 Slope

(Figure 11)Table shows the hinge state of the fixed base building resting on 15◦ sloping ground. The total hinges formed are 1904.The hinges are formed when the displacement has reached 313.67 mm at LS state. No hinges are formed at any other state.Figure shows the final hinge formation at IO state of the building with flexible base which is resting on 15◦ sloping ground.

TABLE: Base Shear vs Monitored Displacement

Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	1904	0	0	0	0	1904	0	0	0	1904
1	31.776	429.33	1900	4	0	0	0	1904	0	0	0	1904
2	69.417	901.536	1800	104	0	0	0	1904	0	0	0	1904
3	105.37	1138.87	1692	212	0	0	0	1904	0	0	0	1904
4	145.89	1278.09	1620	284	0	0	0	1904	0	0	0	1904
5	313.67	1490.47	1536	368	0	0	0	1888	16	0	0	1904
6	475.08	1595.1	1468	436	0	0	0	1720	184	0	0	1904
7	495.03	1604.99	1456	444	4	0	0	1692	212	0	0	1904
8	495.03	1604.99	1456	444	4	0	0	1692	212	0	0	1904
9	495.03	1604.95	1456	444	4	0	0	1692	212	0	0	1904

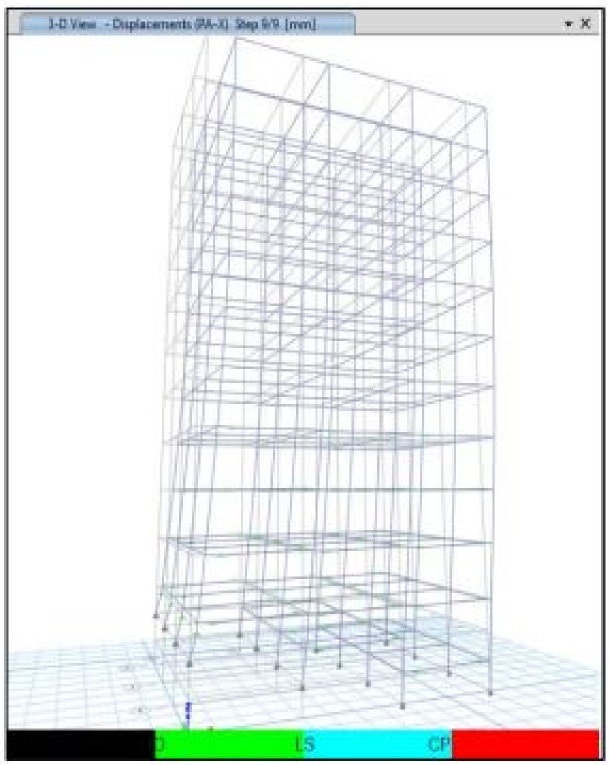


Figure 11: Capacity curve Coordinates for Base Shear and Monitered Displacement with Final Hinge formations

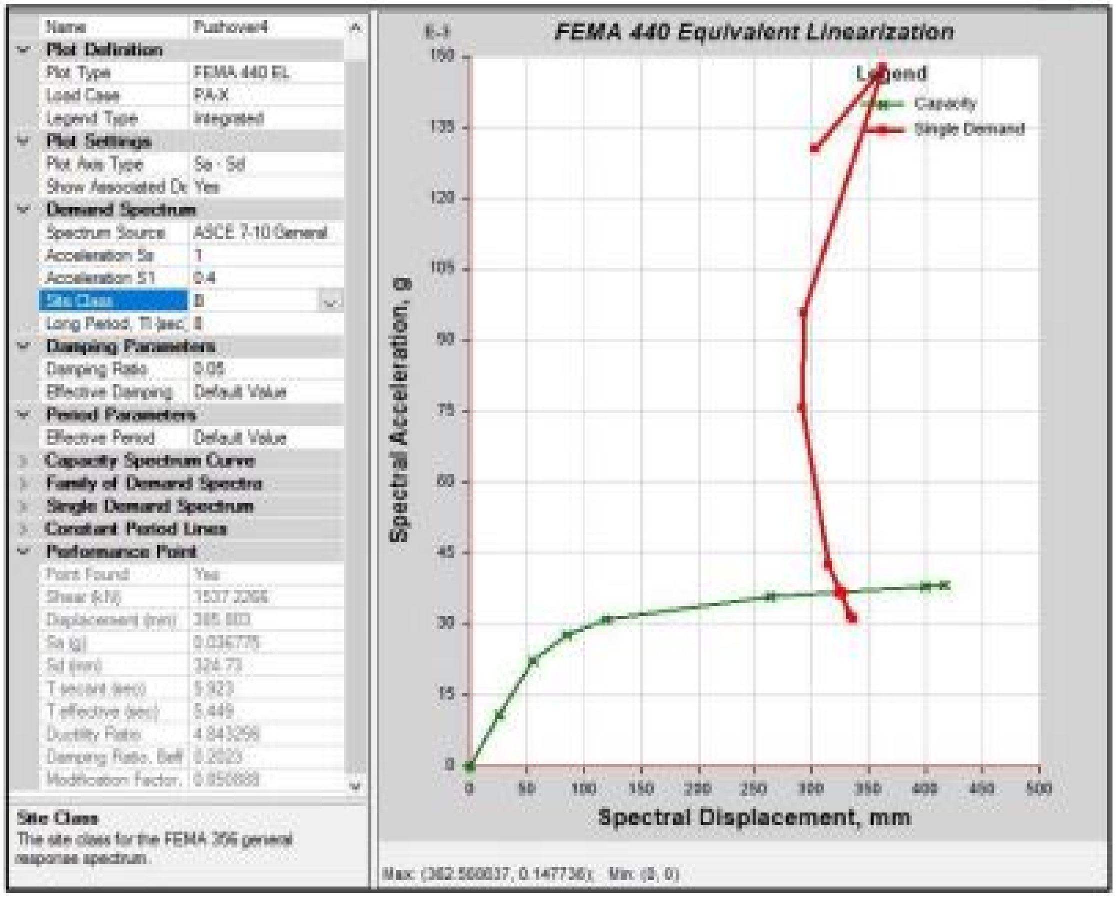


Figure 12: Pushover Curve-FEMA 440 Equivalent Linearization

Figure 12 shows the variation between spectral acceleration (Sa) and displacement (Sd).It is observed that the curve is nonlinear in nature where the curves represent capacity and demand. Performance point is found at 0.036775g spectral acceleration (Sa) and 324.73 mm and spectral displacement (Sd) where the base shear of 1537.2266 KN and displacement 385.803 mm.

4.2.2 27^0 Slope

(figure 13) Table shows the hinge state of the flexible base building resting on 27 ∘ sloping ground. The total hinges formed are 1788.The hinges are formed when the displacement has reached 406.88 mm at LS state. The hinges are formed beyond CP when the displacement has reached 481.32 mm. Figure shows the final hinge formed beyond CP state, located at extreme left base columns of the building with flexible base which is resting on 27∘ sloping ground.

TABLE: Base Shear vs Monitored Displacement

Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	1788	0	0	0	0	1788	0	0	0	1788
1	41.378	649.9642	1780	8	0	0	0	1788	0	0	0	1788
2	86.857	1223.542	1656	132	0	0	0	1788	0	0	0	1788
3	179.49	1611.798	1484	304	0	0	0	1788	0	0	0	1788
4	239.02	1759.259	1440	348	0	0	0	1788	0	0	0	1788
5	406.88	1971.704	1368	420	0	0	0	1684	104	0	0	1788
6	570.64	2120.334	1324	464	0	0	0	1544	244	0	0	1788
7	573.59	2122.534	1324	460	4	0	0	1544	244	0	0	1788
8	481.32	573.2052	1324	456	4	4	0	1544	236	0	8	1788

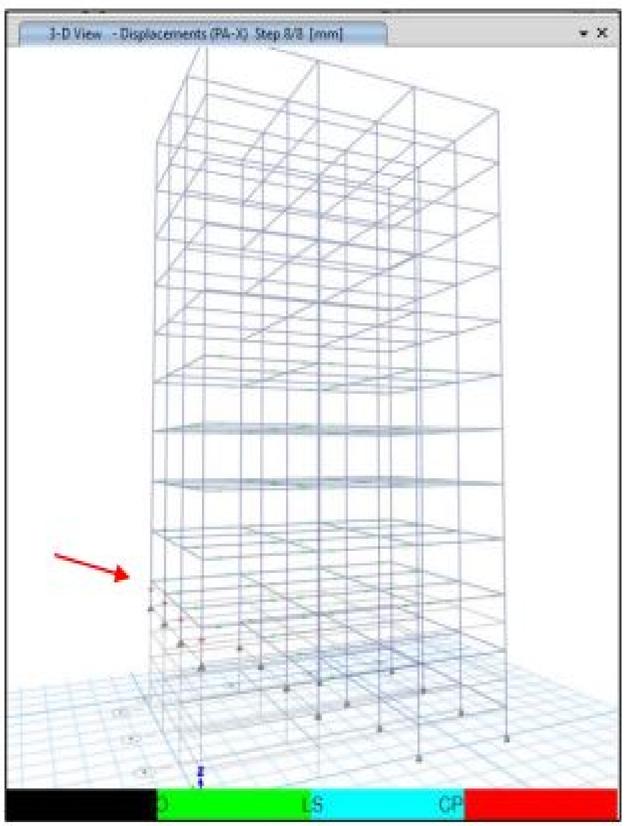


Figure 13: Capacity curve Coordinates for Base Shear and Monitered Displacement with Final Hinge formations

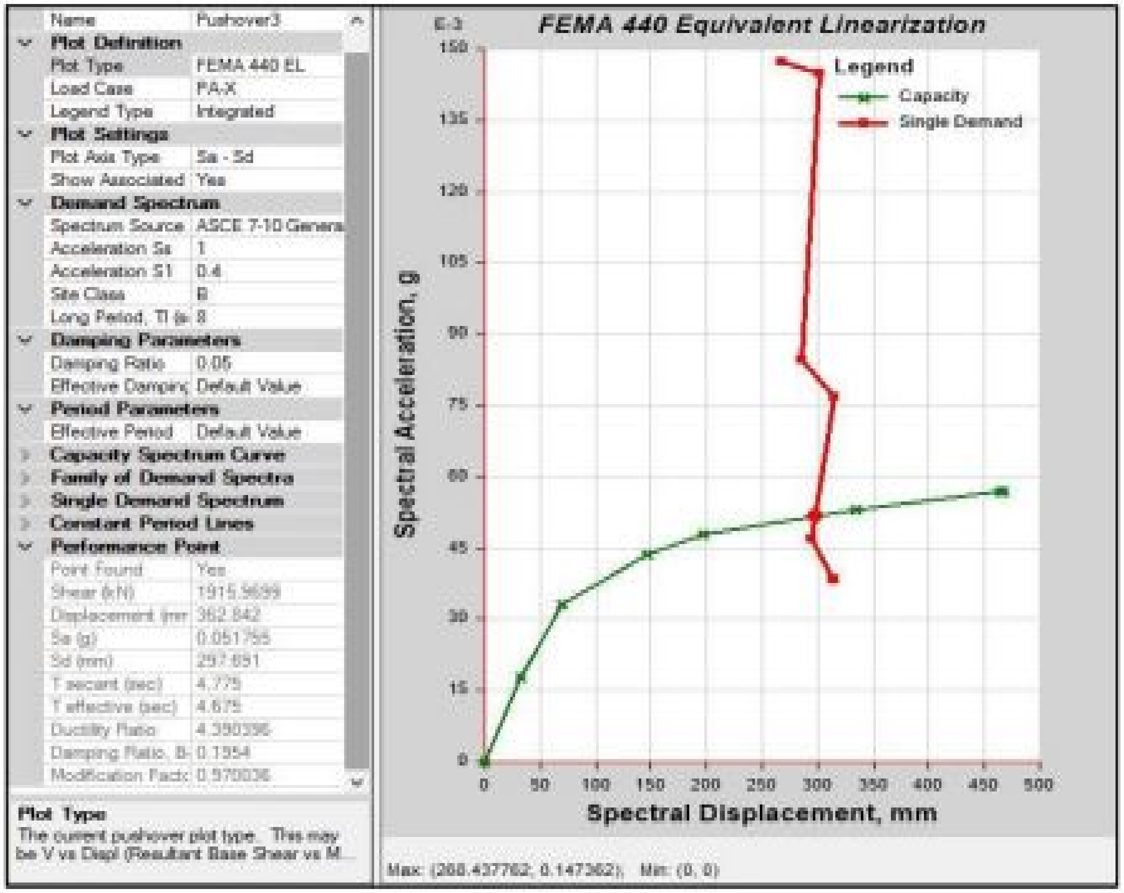


Figure 14: Pushover Curve - FEMA 440 Equivalent Linearization

figure 14 shows the variation between spectral acceleration (Sa) and displacement (Sd).It is observed that the curve is nonlinear in nature where the curves represent capacity and demand. Performance point is found at 0.051755g spectral acceleration (Sa) and 297.691 mm and spectral displacement (Sd) where the base shear of 1915.96 KN and displacement 362.84 mm.

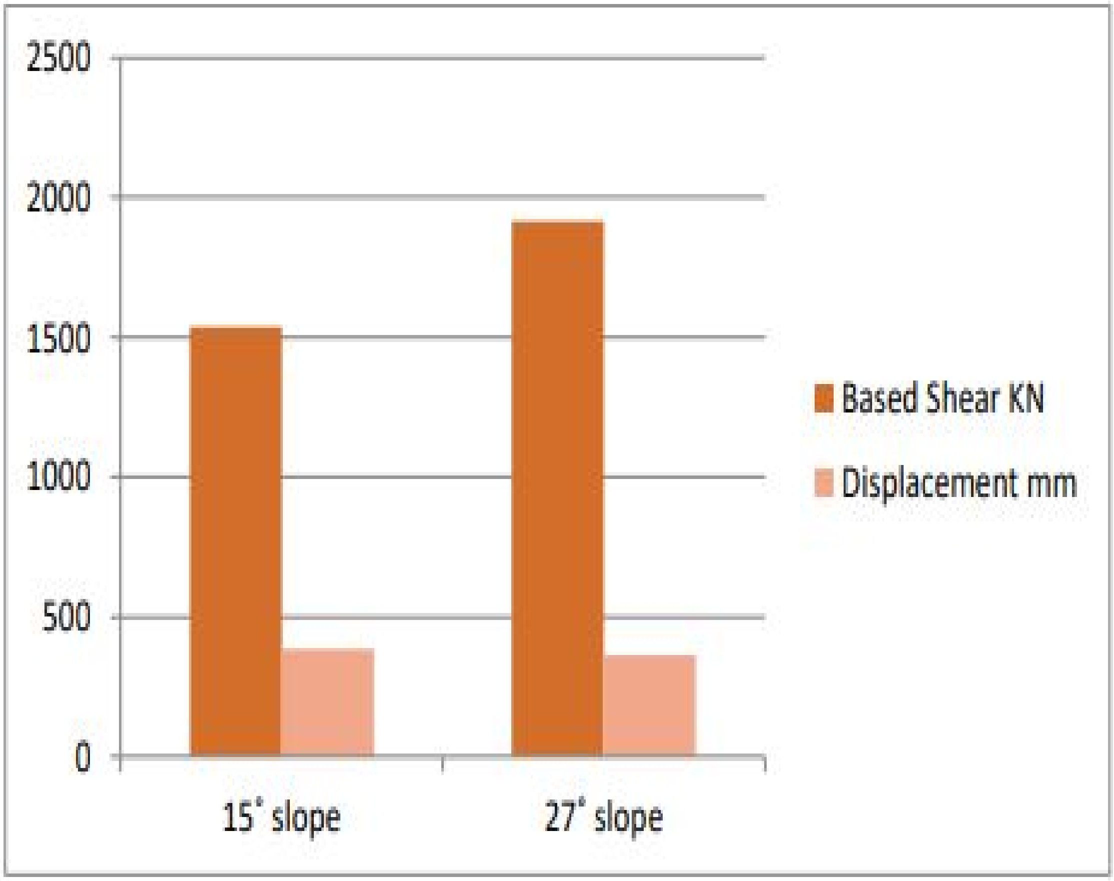


Figure 15: Base Shear and Displacement comparison of Flexible Base System

The figure 15 shows the variation between base shear and displacement v/s angle of slope of the flexible base building structure resting on 15◦ and 27◦. The Base Shear of 15◦ and 27◦ are 1537.22 KN and 1915.96 KN and the Displacements are 385.803 mm and 362.84 mm respectively

Base Shear	Fixed	Flexible
15° slope	1933.7	1537.22
27° slope	2182.78	1915.97

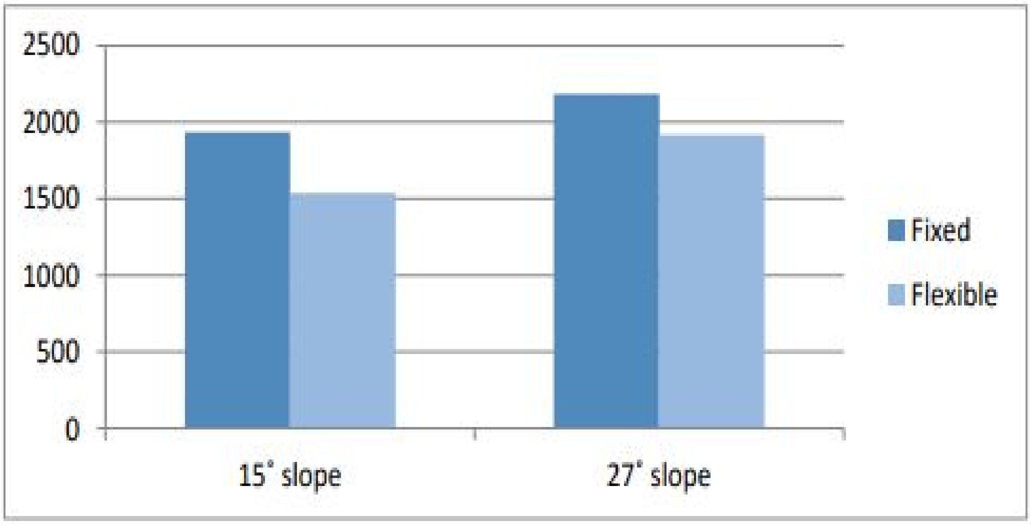


Figure 16: Base shear comparison between Fixed and Flexible system

The figure 16 show the variation between base system and angle of slope. Base shear decreased for flexible base system by 20.50% and 12.22% for 15◦ and 27◦ slope respectively.

5 Conclusions

1. We can observe that in fixed base system, base shear has increased by 11.42% and displacement decreased by 10.28% for 27◦ slope where as in flexible base system, base shear has increased by 19.76% and displacement decreased by 5.95% for 27◦ slope.

2. Base shear decreased for flexible base system by 20.50% and 12.22% for 15◦ and 27◦ slope respectively.

3. For 15◦ slope fixed base 1622 hinges are formed in IO state, 278 hinges in LS state and 4 beyond collapse range and for flexible base 1692 hinges are formed in IO state, 212 hinges in LS state.

4. For 27◦ slope fixed base 1360 hinges are formed in IO state, 272 hinges in LS state, 148 hinges in CP state and 8 beyond collapse range and for flexible base 1544 hinges are formed in IO state, 236 hinges in LS state and 8 beyond collapse range state.

5. We can conclude that base shear increases with increase in slope in both base systems.

6. Base shear is less for flexible base system compare to fixed based system.

7. The maximum hinges beyond collapse range are formed in 27∘ slope with fixed base system.

References

[1] B.G. Birajdar and, S.S. Nalawade. (2004) Seismic analysis of buildings resting on sloping ground. 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada, August 1-6, Paper No. 1472.

[2] S.M.Nagargoje and K.S.Sable. (2012) Seismic performance of multi-storeyed building on sloping ground. S.M.Nagargoje et al./ Elixir Elec. Engg. 53 - 11980-11982.

[3] Kiran.T and N Jayaramappa. (2017) Seismic Performance of RC Frame Buildings Resting on Sloping Ground. IOSR Journal of Mechanical and Civil Engineering.

[4] Santhosh.D. (2014). Pushover analysis of RC frame structure using ETABS 9.7.1. IOSR Journal of Mechanical and Civil Engineeringe-ISSN: 2278-1684.

[5] Nima Sthapit and Nisha Sthapit (2021) Retrofitting of an RC frame building damaged in “April 2015 Gorkha earthquake” in Kathmandu valley Progress in Disaster Science 11 - 100192.

COMPARITIVE STUDY OF CONVENTIONAL SLAB SYSTEM AND FLAT SLAB SYSTEM WITH AND WITHOUT SHEAR WALL USING ETABS

Dr. Vijayalakshmi Akella[1], Mr. Jagadeeshan Y^2,

1. Professor and Head : Department of Civil Engineering, K S School of Engineering & Management, Bangalore, India

3. P G Student: Department of Civil Engineering, K S School of Engineering & Management, Bangalore, India

jagadeesh9727@gmail.com

Abstract

The uses of flat slab in modern buildings are extremely prevalent because it helps to reduce the weight, expedite construction, and save money. Similarly, conventional slabs have progressed over the time in giving benefits like increased rigidity, increased load carrying capability, safety, and affordability. As age of the advancement got under way, flat slabs practice became quite popular. In this current dissertation, Base Shear, Storey Drift, and displacement parameters were examined in G+11 commercial multi-story buildings with a flat slabs and typical slab. It was investigated how both constructions perform and behave in India's seismic zone III, IV, and V. Industrial and commercial buildings built with the flat slabs method have seen higher displacement than those built with traditional slabs systems. It can be concluded from this that a flat slab with a shear wall provides superior displacement resistance. Displacement also keeps growing as structure's height does. Story drift has been observed to be most pronounced for flat slab without shear wall compared to the beam slab system and to least pronounced for the flat slab with Shear wall. The typical slab buildings have stiffer story than flat slab construction. The stiffness keeps getting worse as story progresses.

Keywords:

Earthquake, Beam slab system, Flat slab system, Shear wall, Base shear, Maximum storey displacement, Maximum storey drift

1 Introduction

Beam Slab System (Framing System): Beams that frame the columns and supporting pieces that cross between the beams make up this structure. It is an extremely conventional framework. The moderately profound shafts give a solid floor able to do long traverses and ready to oppose lateral loads. The frame of this kind of floor has prompted a diminishing as a result of the confusions of beam formwork, coappointment of administrations and general profundity of floor. Slab, beams and columns are the components of framing system. Flat slab System: Flat Slab is a single or double-route structure with drop panels, thickenings at the loadbearing walls and columns along the walls. Drop panels that resemble T-beams are placed over the backings. In order to increase the sparing traversing range, they construct the floor structure's shear limit and structural solidity beneath the vertical loads. As of late this type of development has turned out to be less mainstream, in light of the point of confinement on practical ranges of around 9.5 m. Shear wall System: The vertical components of a structure that withstand horizontal forces are called shear walls. Along their entire length, shear walls directly withstand lateral forces.

Detailed longitudinal and transverse reinforcement can provide the necessary strength to stop structural damage during earthquakes. The structure is subjected to lateral forces that are exerted horizontally by winds and earthquakes. Lateral forces, which are those that are applied to a building horizontally and are brought on by an earthquake or wind, can cause a wall to shear or topple. The earthquake can induce catastrophic damage which is the major hazards. Its direct effect includes ground movement and faulting. Its indirect effect damages indirectly. This paper gives a clear idea of comparative behavior of Conventional slab system, Flat slab system and Flat slab sytem with shear wall. Structural parameters like Time Period, Base Shear Absorption, Maximum Storey Displacement and Maximum Storey Drift are studied for different Structural systems (Conventional slab, Flat slab &Flat slab with shear wall) for different earthquake zones as shown in the Figure 1

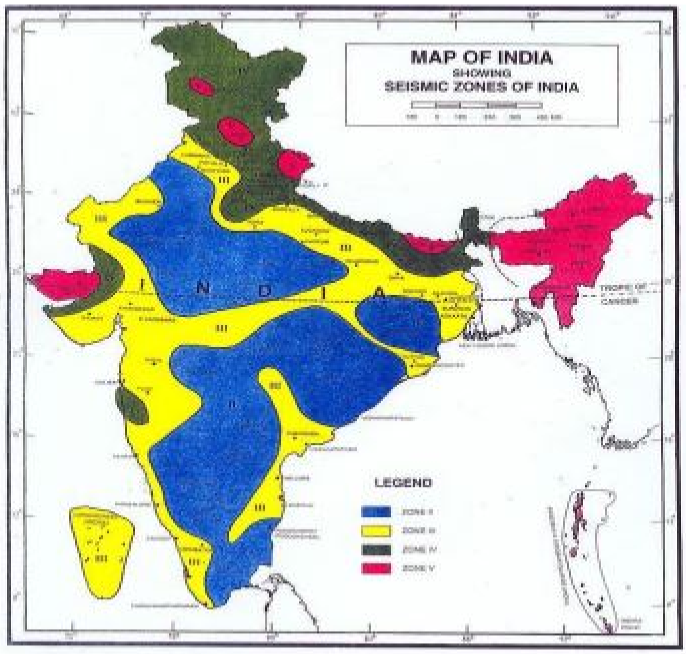


Figure 1: Recent map Indicating Sesmic Zones of India (IS-1893 (Part 1):2016)
(Source : Geological Survey of India (GSI))

2 Literature Review

Medium-high rise buildings at different location were considered. Five models of twelve storey buildings are considered in zone III by changing various location of shear wall. Model one was bare frame and remaining 2 four models with shear wall at different positions. The models were analysed by linear static method Parameters like storey drift, storey shear and displacement were compared and calculated by using ETABS. It was concluded that it was more effective providing shear walls to resist lateral forces. [1].

To determine the shear wall position in multi-story buildings based on both in elastic and elasto-plastic behaviour. Consideration was given to the 15-story building which was located in Seismic Zone IV. We looked at two models, one with and one without a shear wall. STAAD Pro and SAP 2000 software were both used to conduct elastic and elasto-plastic analyses. In both situations, shear force, bending moment, storey drift, and the location of the shear wall were estimated. Accordingly, it was claimed that shear walls might be added in the shorter direction between the sixth and seventh or the first and the twelfth frames. [2] .

In order to determine the best location for the shear wall in multi-story building, five models of 25-story buildings in seismic zone V were taken into consideration. One model had a bare frame, and the others had shear wall models in various locations. The models were examined using linear static and linear dynamic methods, as well as the impact of the building's central concrete core wall. ETABS was used to determine variables such as displacement, storey shear, and storey drift. The study's concludes that Model 5 performs better than other models. [3]

The fix for the shear wall's placement when evaluating seismic performance, RC-framed buildings with six, twelve, twenty-four, and thirty-six stories and at different position of shear wall were taken into account. Eight models in total were taken into account. Model 1 had an in-filled frame but no shear walls, while the other versions assigned with shear walls. To determine the parameters such as time period, lateral displacement, damping, and base shear by using ETABS, the seismic performance evaluation was carried out using the response spectrum or elastic analysis as well as nonlinear static pushover or in-elastic analysis. It was determined that model 7 and 8 exhibit improved lateral load characteristics as well as increased lateral stiffness. [4]

Six-story structure under consideration in seismic Zone IV was examined using various shear wall forms and positions. In order to calculate characteristics like axial force and moments in the Y and Z directions, four models were taken into consideration, with one being bare frame and the other three being shear walls of the same length at various locations. STADD-pro analytic software was used to complete the analysis. It was determined that the presence of a shear wall at a different position had an impact on the axial stress on the column. [5]

3 Objectives:

The main objective of this study is to analyze the seismic behaviour of Conventional Beam Slab system, Flat slab system and Flat slab system with shear wall by comparing various design parameters like Time Period, Base Shear Absorption, Maximum Storey Displacement and Maximum Storey Drift

4 Methodology

JReferred journal publications, manuals, conference publications, case studies etc.is surveyed and reviewed to understand the behaviour of different structural systems. The number of models with different structural systems for different seismic zones are modelled using ETABS version 18.1.1 software by considering G+11 stories to study the seismic behaviour offthe structure. The different model configurations considered are; Beam slab systems, Flat slab systems and Flat slab system with shear wall. The equivalent Static analysis and response spectrum analysis are carried out on the models of RCC building with different structural systems for different seismic zones as per IS 1893 (Part-1): 2016 [6] to study the lateral load resisting capacity and to observe the seismic behaviour of conventional slab system and flat slab system with and without shear wall for different seismic zones. Modal analysis is carried out for all the models as per IS 1893 (Part-1): 2016, to get time period, natural frequency and mode shapes. In this study, the modal analysis is carried out with different number of modes in the increasing order from 12 modes to 20 modes to check the codal provisions of modal participating mass ratios like translation along X and Y directions and rotation along Z direction for different number of modes. Parametric studies are carried out for all the models as perIS1893 (Part -1): 2016, to study the lateral load resisting capacity and to observe the seismic behavior of RCC building in considered three different seismic zones. The design parameters that are considered for the study are Base shear absorption, Maximum storey displacement, and Maximum storey drift.[7, 8, 9]

5 Results and Discussions

The structural response of multi-storey building with Conventional Beam slab system, Flat slab system and Flat slab system with shear wall for different seismic zones is carried out. In this study the Response Spectrum analysis results from the three structural systems for different seismic zones were compared and analysis results obtained using ETABS software is represented in the form offgraphs. The important parameters that are considered in the comparison are:

- ➢ Time period
- ➢ Bases shear absorption
- ➢ Maximum storey displacement
- ➢ Maximum storey drift

5.1 Time Period

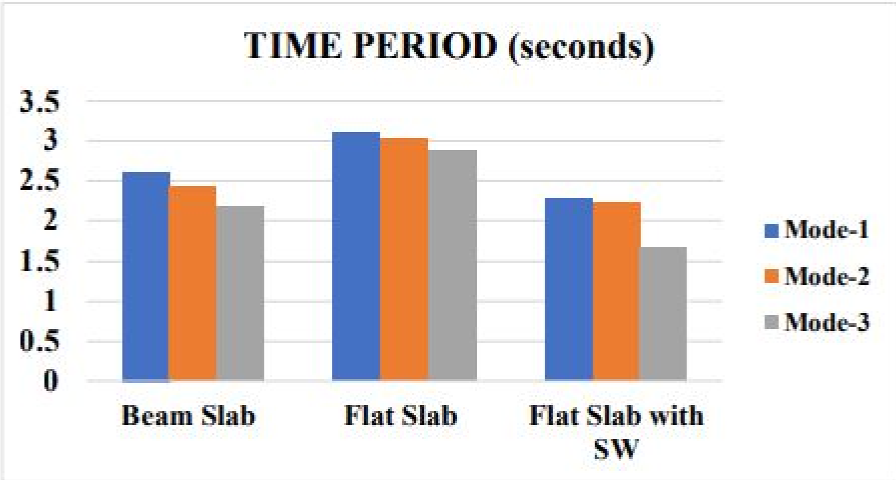


Figure 2: Time period for three different structural systems in Seconds

Figure 2 displays the Time Period in seconds for the first three modes for three different structural systems and shows that the time required for the Beam Slab & Flat Slab system with shear wall is shorter than for the Flat Slab system without shear wall. In comparison to Beam Slab and Flat slab systems without shear wall systems, stiffness is attained greater with the Flat slab system with shear wall because offthe additional shear walls.

5.2 Base Shear

The present study compares base shear, which involves comparing the base shear measurements made for various structural systems in order to track base shear fluctuation throughout various zones. For various zones, the bar graph depiction of base shear variation for various structural systems in EQX and EQY direction is shown in Figures 3 to Figure 5

5.2.1 Zone -III

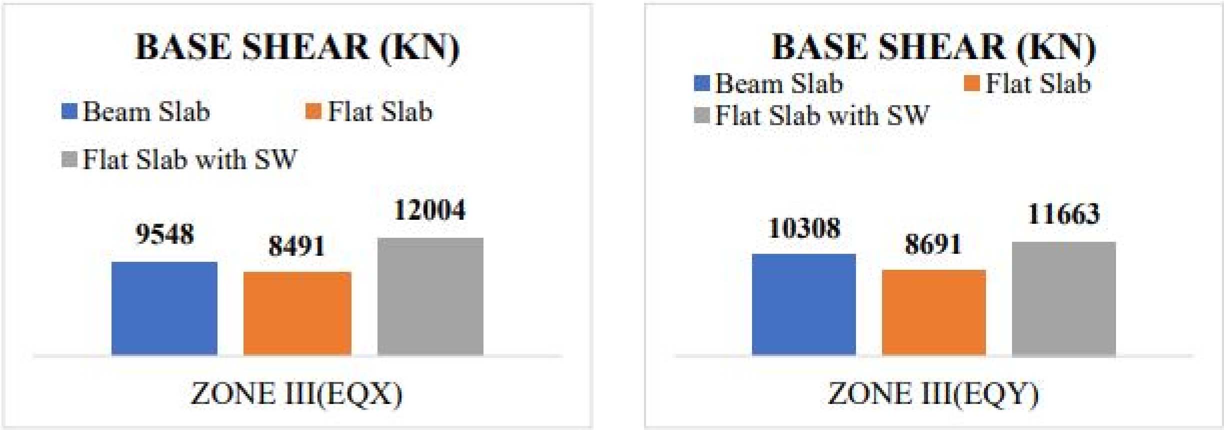


Figure 3: Base Shear(KN) in Zone III for EQX and EQY

5.2.2 Zone-IV

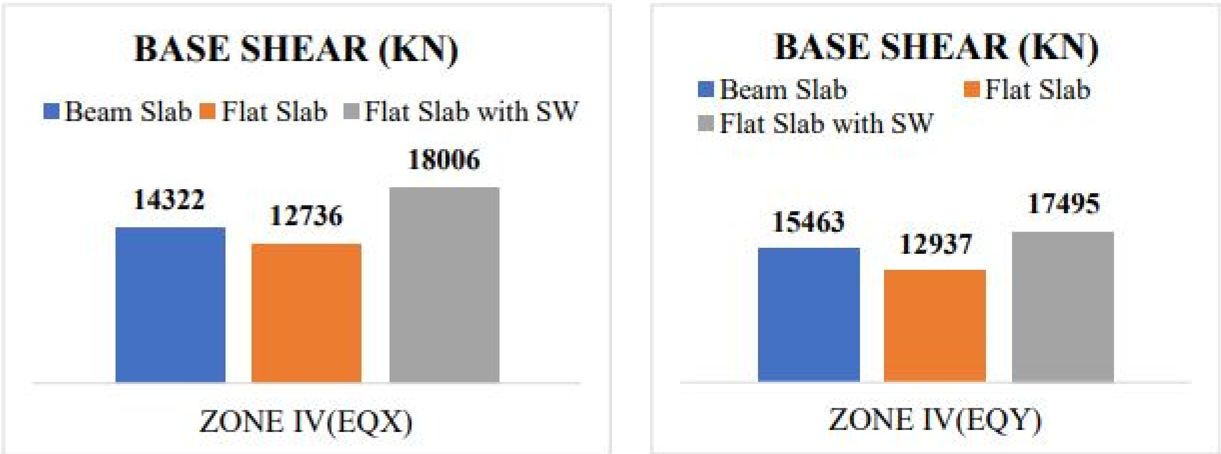
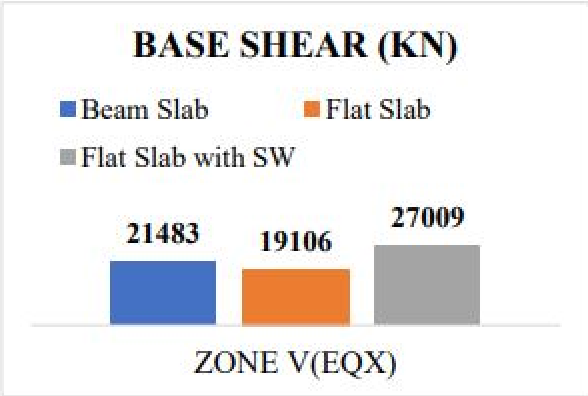


Figure 4: Base Shear(KN) in Zone IV for EQX and EQY

5.2.3 Zone -V



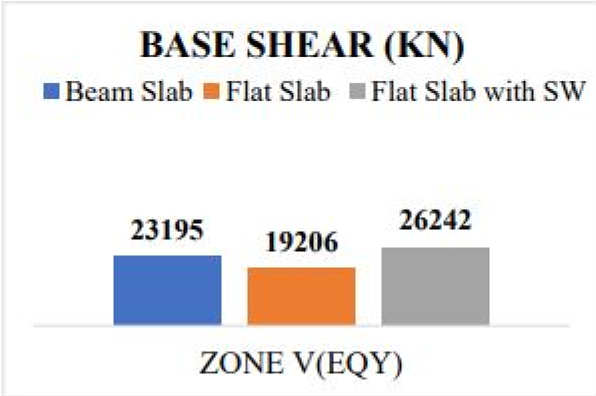


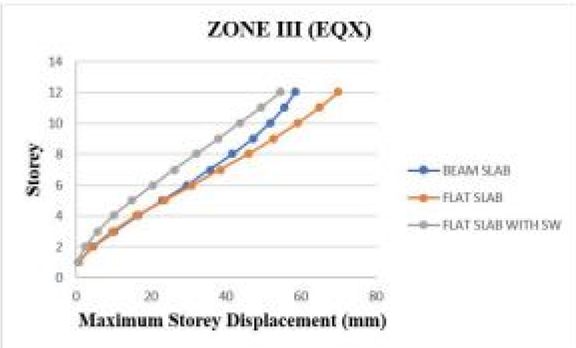
Figure 5: Base Shear(KN) in Zone V for EQX and EQY

Figure 3 - Figure 5 demonstrates the bar graph representation of base shear variation for three distinct structural systems for seismic zone III to IV. More base shear results from greater seismic mass. Base shear is influenced by the building's proportions in the x and y directions.

5.3 Maximum Storey Displacement:

A storey displacement is defined as the locations where the maximum storey displacement occurred. The lateral displacements off three separate seismic zones for all the different structural systems for G+11 storey building models subjected to X and Y directions is analyzed and the same is represented in the form of graphs which is shown in Figure 6 - figure 8

5.3.1 Zone -III



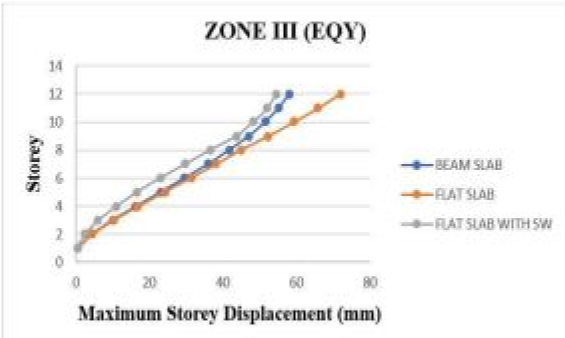


Figure 6: Maximum Storey Displacement in Zone III for EQX & EQY

5.3.2 Zone -IV

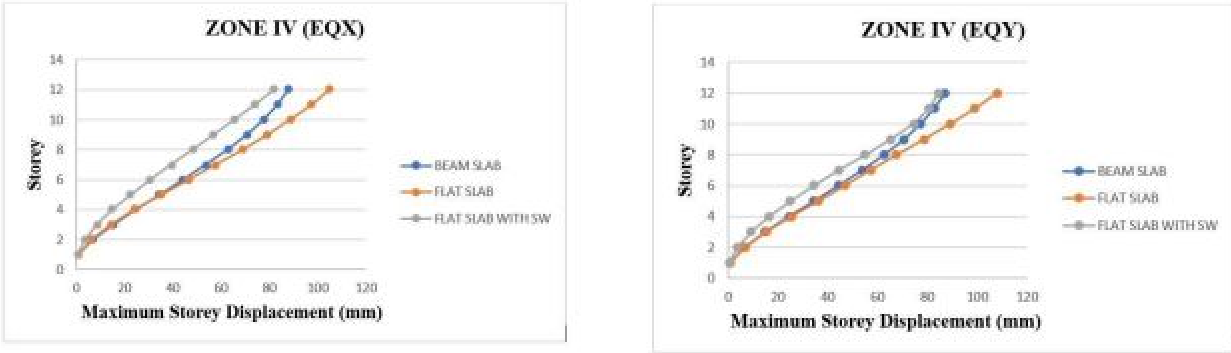


Figure 7: Maximum Storey Displacement in Zone IV for EQX & EQY

5.3.3 Zone -V

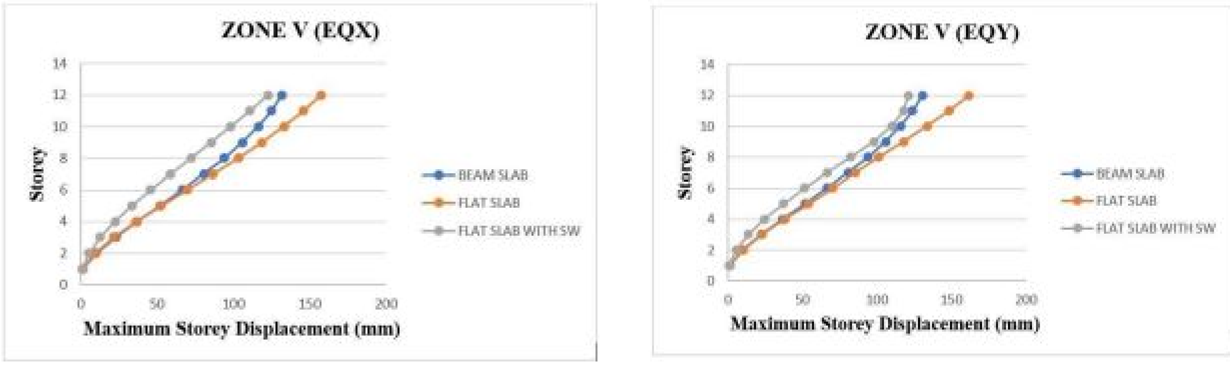


Figure 8: Maximum Storey Displacement in Zone V for EQX & EQY

5.4 Maximum Storey Drift:

Storey drift is how lateral displacement works. Storey drift is taken into account at the intersections of the beam and column. In this study, the maximum drifts that happened at each of these sites are compared.

5.4.1 Zone -III

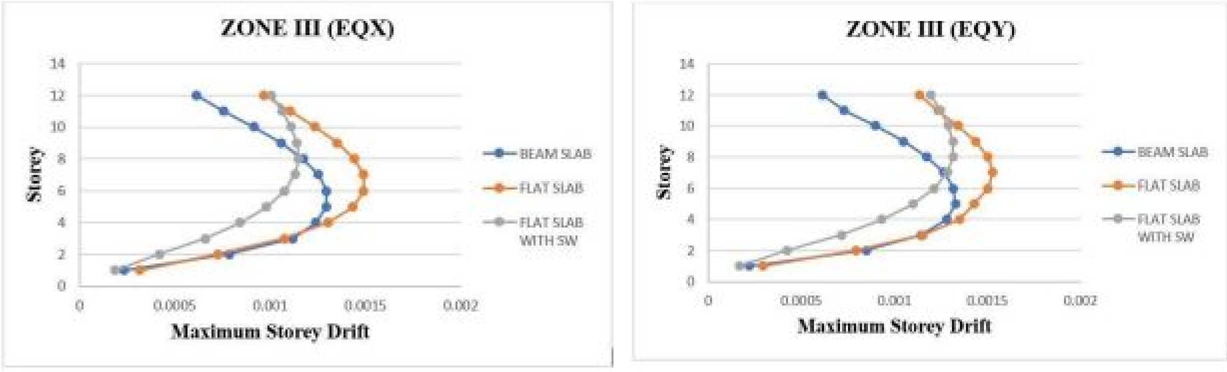


Figure 9: Maximum Storey Drift in Zone III for EQX & EQY

5.4.2 Zone -IV

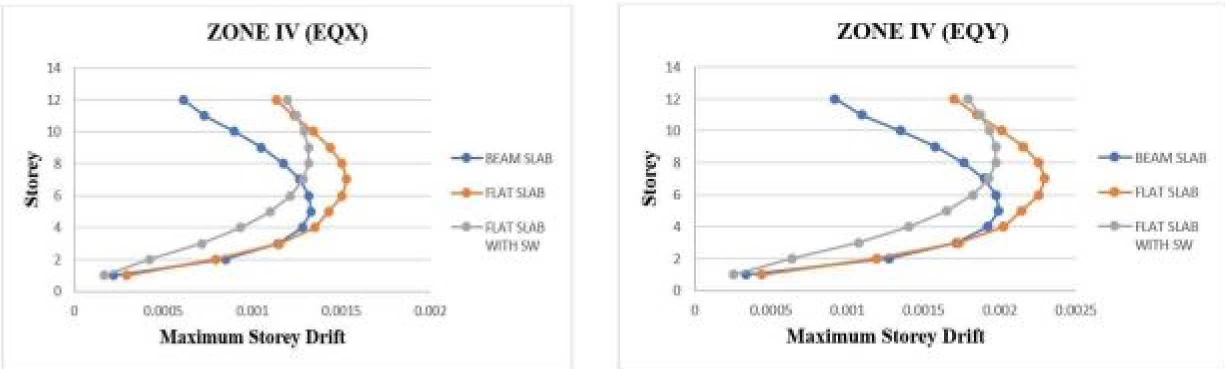


Figure 10: Maximum Storey Drift in Zone IV for EQX & EQY

5.4.3 Zone -V

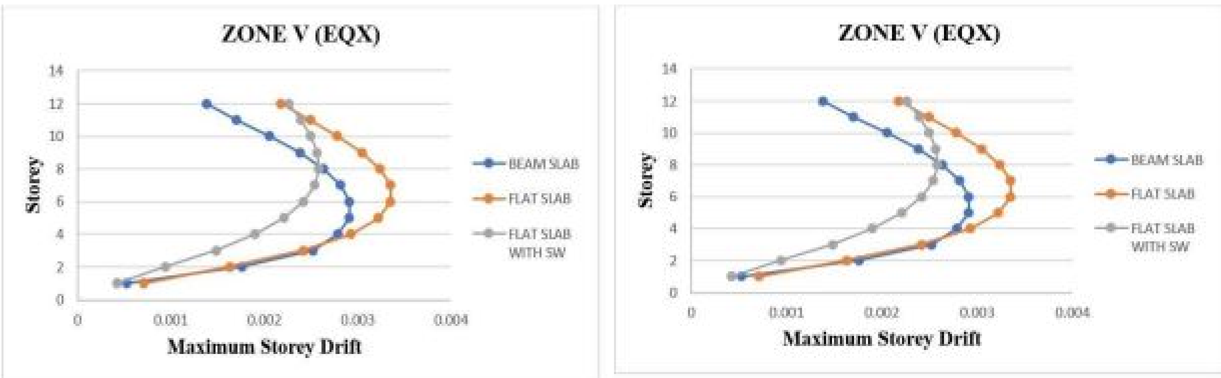


Figure 11: Maximum Storey Drift in Zone V for EQX & EQY

Figure 9-Figure 11 illustrates the graphical depiction of the variation of storey drift for the three structural systems for seismic zone III to zone V in the direction EQX & EQY.

6 Conclusions

The current study compared three structural systems and evaluated the seismic performance of reinforced concrete multi-story buildings. A 12-story structure that takes into account three separate structural systems for three distinct seismic zones is considered. Medium soil type (II) and seismic zones III, IV, and V are taken into consideration in as per IS1893 (Part-1): 2016. To verify the results of time period, base shear absorption, maximum storey displacement, and maximum storey drift, the equivalent static and response spectrum analysis of the models has been carried out using ETABS software. The conclusions of the study are as follows;

1. Modal analysis is carried out for all three models to obtain the time period, natural frequency and mode shapes. Time period is less in flat slab system with shear wall compared to beam slab system and flat slab system. Time period is less compared to flat slab system of about 16% and 27% of beam slab system and flat slab system with shear walls respectively.

2. Equivalent static analysis and Response spectrum analysis was carried out for all the three models for different seismic zones, to get Base shear, Lateral displacement and storey drift. As the stiffness is more in Flat slab system with shear wall Base shear is more compared to beam slab system of about 20% in EQX direction and 12% in EQY direction. Similarly, when compared with Flat slab system stiffness is more in Flat slab with shear wall by 30% in EQX direction and 25% in EQY direction.

3. As the flexibility is more in Flat slabs system compared to beam slab system and flat slab system with shear wall, storey displacement is more compared to flat slab system with shear wall of about 16% in EQX direction and 19% in EQY direction. When compared with beam slab system it is more by 22% in EQX direction and 24% in EQY direction. Due to the behaviour of less framing action in flat slab system displacement is more.

4. Storey drift is the displacement of two stories to the height of one storey; hence Storey drift is the function of lateral displacement. If the number of stories increases, lateral displacement increases, similarly storey drift also increases. Beam slab system and Flat slab system with shear walls shows less storey drift because displacement is less. The storey drift is less in Flat slab system with shear wall compared to Flat slab system of about 20% in EQX direction and 18% in EQY direction. When compared with beam slab system the storey drift in Flat slab system with shear wall is less by 15% in EQX direction and 14% in EQY direction.

5. Storey drift for all models of three models for three different seismic zones are within the permissible limits as per.

References

[1] C. G. Quintero, "Behavior of Slab-Band Floor Systems subjected to Lateral Earthquake Loading," Eleventh World Conference on Earthquake Engineering, pp. 462 - 471, 1996.

[2] G. V. S. Himaja, "Comparative Study on Non-Linear Analysis of Infilled Frames for Vertically Irregular Buildings," International Journal of Engineering Science Invention (www.ijse.org), vol. 4, no. 6, pp. 44 - 51, June 2015.

[3] L. M. R. A. M. J.Sagaseta, "Punching of flat slabs supported on rectangular columns," Journal of Engineering Structures, vol. 77, pp. 17-33, 2014.

[4] R. S. More, "Analysis of Flat Slab," International Journal of Science and Research (www.ijse.net, vol. 4, no. 7, pp. 98 - 101, July 2015.

[5] Chaitanya, "Analysis of Multi-Storey Building with Precast Load Bearing Walls," International Journal of Civil and Structural Engineering, vol. 4, no. 2, pp. 147 - 157, 2013.

[6] "IS 1893(Part-1): 2016 "Criteria for Earthquake Resistant Design of Structures" BIS, New Delhi".

[7] "IS: 875 (Part 1) - 1987 , Code of practice for design loads for buildings and structures, Part 1 - Dead load"

[8] "Bureau of Indian Standard, IS-456(2000), Plain and Reinforced Concrete Code of Practice".

[9] "IS: 875 (Part 2) - 1987 , Code of practice for design loads for buildings and structures, Part 2 - Imposed load"

PLANNING SRATEGIES FOR PERI URBAN AREA DEVELOPMENT NEAR PIMPRI CHINCHWAD MUNICIPAL CORPORATION

Deepali S Patil[1], Dr. Rama R Subrahmanian [2]

1. Practicing Architect, Associate professor MVIT, Bangalore, India

2.Principal Dayananda Sagar College of Architecture, Bangalore, India

Abstract

The increase in urbanization, emerging economical activities and land speculation has led to the formation of peri-urban settlements around the Indian Megacities. The characteristics of these settlements are changes in land use and occupation pattern, reduced farm activities, growth of built structures, land agglomeration, etc. There is an inadequate planning and governance by the local government in these peri-urban regions. This brief calls on the Indian government to formulate a broad policy for planned spatial growth of megacities to ensure the sustainable development of the country's peri-urban areas. The study area forms a part of the Pune district in the state of Maharashtra (India). Pimpri Chinchwad is a relatively newly developed urban area of Pune city. The basic idea behind the creation of these twin cities was to reduce pressure on the resources located in the Pune core area. The transformation of urban fringe areas of the city in an unsustainable manner is a special challenge for urban planning. The paper focuses on facts and figures of peri urban areas around Pimpri-Chinchwad, the reasons behind its formation, the issues of these fringe areas and approaches to overcome them. Environmental planning and management of the peri-urban areas cannot be based on conventional planning approaches in urban and rural areas but also on an approach that responds to the specific environment, social, economic and institutional aspects of the peri-urban interface.

Keywords

Peri urbanization, Fringe zone development, Land use, and socio-economic development of Pimpri Chinchwad.

1 Introduction

In most Metropolitan cities in India, the urban population increases very rapidly because of socioeconomic activities within the urban area, better job opportunities, education, transportation, housing, access to the basic needs and better quality of life. As a result, productive agricultural land gets converted to the urban land use. Due to the unplanned and haphazard development in the peripheral areas, infrastructure facilities and basic social amenities are to be provided for these new settlements. There is a lack of proper guideline and monitoring systems at the institution level of Peri-Urban Area (Villages) as comparative to strong Development Control Regulations, Planning Guidelines and Monitoring System in the urban areas. This attracts people and developers to concentrate or to invest in these areas, particularly in the immediate peripheral area of the urban limit. The study area forms a part of the Pune district in the state of Maharashtra (India). Pimpri-Chinchwad Municipal Corporation is situated on Mumbai Pune National highway. It is located to the North-West of Pune and is well connected to the Pune city proper via the old Pune- Mumbai Highway (fig. 1).

Figure 1: Pimpri Chinchiwad Location map

Pimpri Chinchwad is a newly developed urban area of Pune city. Due to availability of water, land and transportation resources, the twin cities emerge as an industrial hub. The basic idea behind the formation of these twin cities was to reduce pressure on the resources located in the Pune core area. Industrialization in Pimpri area commenced with the establishment of Hindustan Antibiotics Limited in 1956. The establishment of the Maharashtra Industrial Development Corporation (MIDC) in 1961-considerably facilitated the industrial development in the area. The Municipal Council established in 1970, incorporating four village-panchayats in the area. In 1982 the civic body was upgraded to its present status as “Municipal Corporation”. Its population has increased from 85,000 in 1971 to 1,730,000 as on today. There are about 4000 industries under the corporation area. TELCO, Bajaj Auto, SKF are some of the major companies. Pimpri Chinchwad lies between 18° 25’ to18° 40’ North latitude and 73° 45’ to 73° 50’ East longitude with an area of about 170.51 sq.km. It is composed of 64 wards according to 2012 which comes under 4 administrative divisions of Pimpri Chinchwad Municipal Corporation (PCMC). The soil in this area is generally brownish copper colored in the west and somewhat blackish in the east. Normally all along the river side there is fertile soil suitable for agriculture. In last 10 years, the population of PCMC is growing very fast. Population of Pimpri Chinchwad as per census 2011 is 1,729,320 of which 945,914(54.70%) are males and 783,406(45.30%) are females with a gender ratio of 828 females per 1000 males. Pimpri Chinchwad has an average literacy rate of 87.19, higher than the national average of 74.04%. This increasingly growth rate of population is responsible for increasing the demands of residential land. Consequently population growth is one of the important factors in changing the urban land use of PCMC. Population growth of Pune Muncipal Corporation(PMC) and PCMC is as represented in figure **??**

Census Year	Population		Decadal Change		Growth Rate	
	PCMC	PMC	PCMC	PMC	PCMC	PMC
1961	39654	794052	----	-----	-----	-----
1971	98572	1029466	58918	235414	148.58	29.64
1981	251769	1202848	153197	173382	155.42	16.84
1991	520639	1566651	268870	361803	106.79	30.24
2001	1006417	2540069	485778	973418	93.30	62.13
2011	1729320	3115431	722903	575362	71.82	22.65

Table 1: Population of PCMC and PMC, Source: Census and PCMC and PMC record

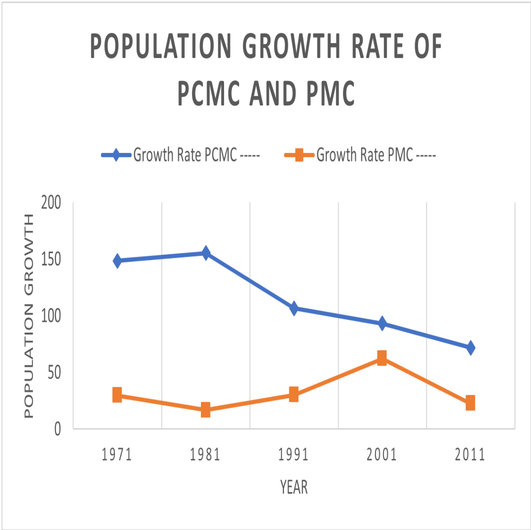


Figure 2: Population growth rate (PMC & PCMC)

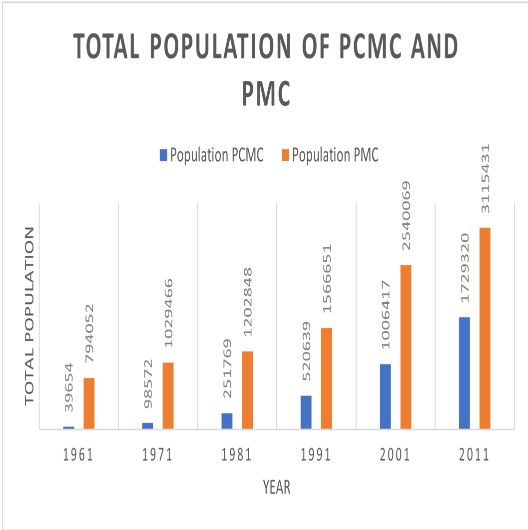


Figure 3: Total Population(PMC & PCMC)

figure 2&figure 3 show population growth rate and Population of PCMC and PMC from 1961 to 2011. The blue line shows the population and growth rate of PCMC and red line shows population and growth rate of PMC. In the last decade, the decadal growth rate of population has been in the range of 70% while the previous three decades witnessed population growth in the range of 30-45%. Between 1951 and 1961 the growth rate of population has been 95%. The population of Pimpri-Chinchwad and Pune as per 2011 Census is 1729320 and 3115431 persons respectively. The current population is approximately estimated to be 41 lakhs. PMC has a population of 2.54 million (2001) which accounts for 35 percent of the total urban population in Pune District and 60 percent of total PMR(Pune Metropolitan Region) population. The PMC's population has grown from 1.57 million in 1991 to 2.54 million in 2001, and in the last decade experienced a compounded annual growth rate (CAGR) of 4.94 percent. PMC's growth is not limited to few but influenced by various factors. It is the most preferred destination for many citizens in Maharashtra for job, education, healthcare treatment, real estate investment; better quality of life etc. as Mumbai is densely populated with high cost of living. Rapid growth of the city however mainly attributed to industrialization of PMC/PCMC after 1960 and expansion of information technology (IT) industry in the last decade. PCMC has a population of 1.01 million (2001) which accounts for 14 percent of the total urban population in the Pune District and 23 percent of the total PMC population. The PCMC's population has grown from 0.52 million in 1991 to 1.01 million in 2001. PCMC has experienced a high CAGR after its industrialization in 1960, almost thrice than that of PMC growth.

2 Expansion of Pune city:

Pune is growing exponentially in terms of its spatial extent (with new surrounding villages being brought under its ambit every ten years or so), its economic activities and its demography. The built-up area in Pune Municipal Corporation (PMC) has expanded about 7.5 times from 18.3 km2 in 1973 to 139.4 km2 in 2013 with an annual rate of expansion 3 km2 per annum (figure 4&figure 5). Between 1973 and 1992, Pune witnessed only 38.5 km2 growth in the built-up areas with 2 km2 per annum Between 1992 and 2013, the city witnessed massive growth of about 82.5 km2 of built-up areas with an expansion rate of

3.9 km2 per annum . If this urban growth trends continue, 67.1 km2 additional built-up area to reach 206.4 km2 in the year 2030 (Figure 2, SLUETH urban growth model simulations)

Figure 4: Expansion of Pune over time

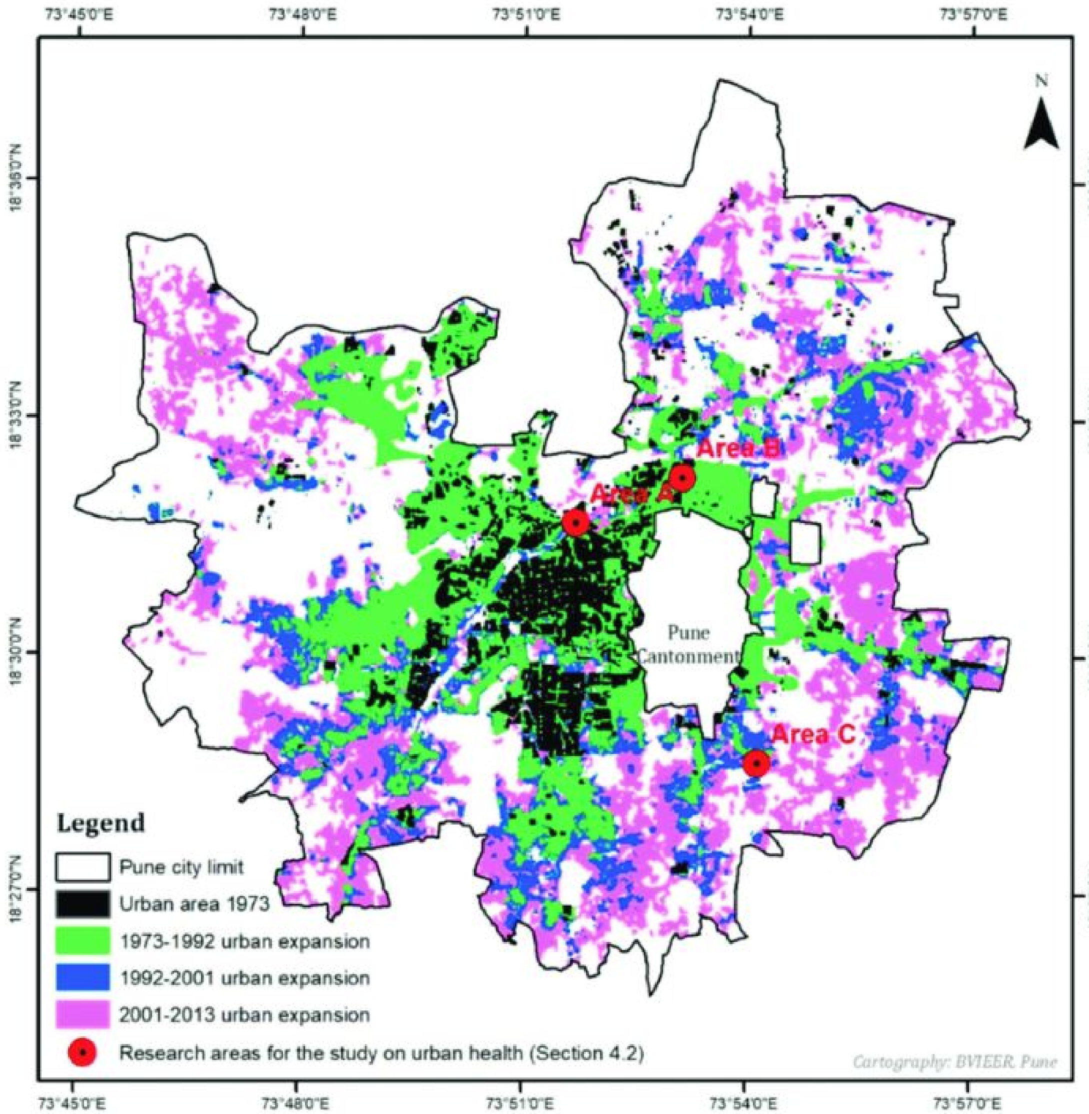


Figure 5: Land use change in PCMC

Land use beginning from 1986 and subsequent distribution provides the physical situation within which land use change occurred. As per the town development plan the total land in the study area is 170.51 sq.km. Among them 32.42 sq.km, 0.59 sq.km, 12. 33 sq.km, 3.89 sq.km, 1.66 sq.km, 0.81 sq.km or 32 % were developed in 1986 used for residential, commercial, industrial, transportation public (table 2)

Land use	Area in Sq.Km		Total	Areas in %
	1986 DP	Newly Merged areas		
Residential	10.16	22.26	32.42	19.01
Commercial	0.40	0.19	0.59	0.35
Industrial	10.85	1.48	12.33	7.23
Public and Semi public	0.84	0.82	1.66	0.97
Public Utilities	0.42	0.39	0.81	0.48
Transport	4.26	2.38	6.64	3.89
Reserved, Forest & Agri	26.27	33.27	59.54	34.92
Water Bodies	1.89	0.95	2.84	1.67
Quarry	0.22	2.46	2.68	1.57
Residential/ Open Spaces	0.69	0.01	0.70	0.41
Barren / Vacant Lands	30.01	20.29	50.30	29.50
Total	86.01	84.50	170.51	100

Table 2: Land use distribution as per development plan, Source: Town planning department, Pune

2.1 Land use pattern in changes from 1973-2011:

The changes from land use from 1973 to 2011 are presented below

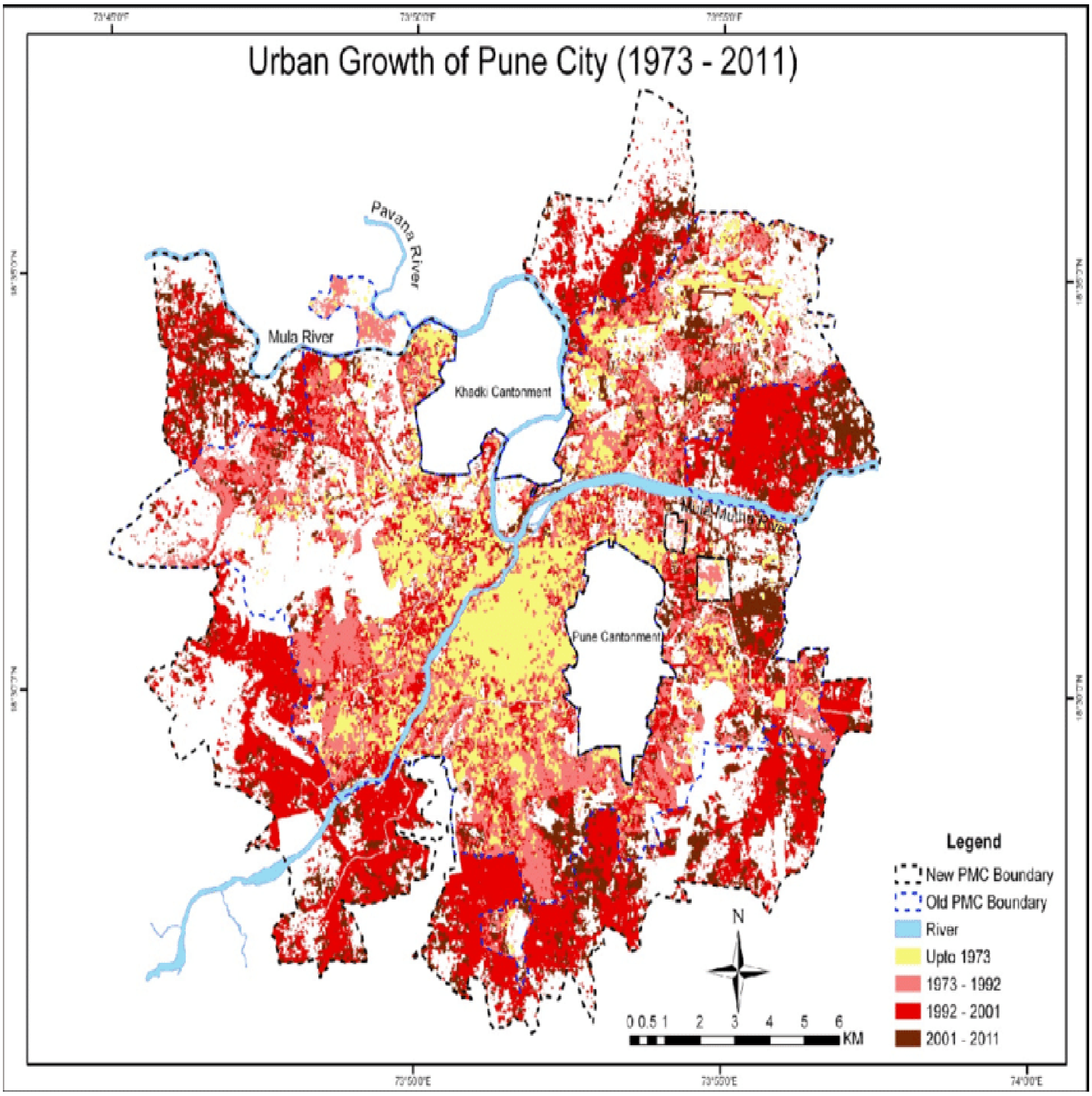
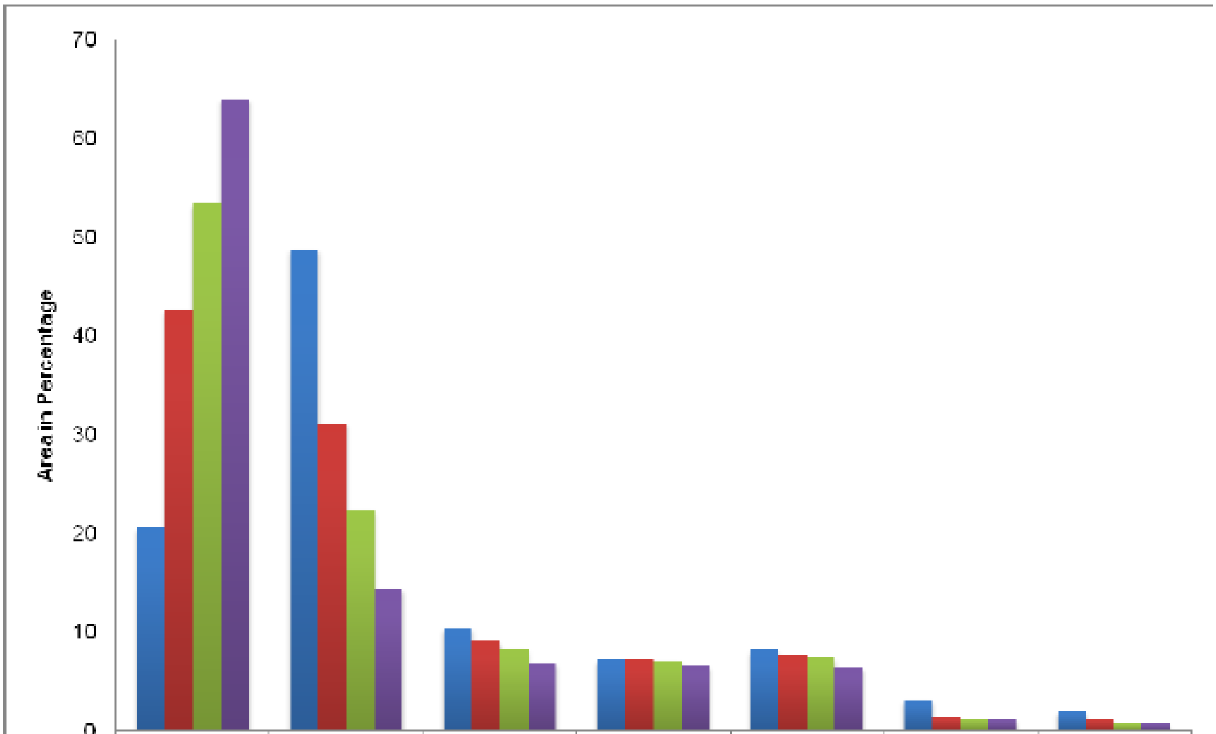


Figure 6: Urban growth of Pune (1973-2011



	Built-up	Scrub Land	Agricultural Land	Fallow Land	Vegetation	Rivers and Lakes	Canal
1973	20.54	48.78	10.39	7.19	8.14	3.09	1.87
1992	42.55	31.23	9.09	7.21	7.62	1.25	1.05
2001	53.33	22.25	8.25	7.05	7.37	1.12	0.64
2011	63.97	14.48	6.90	6.52	6.41	1.09	0.64

Figure 7: Urban growth of Pune (in numbers, 1973-2011)

3 Observations of land use change in PCMC :

The land use pattern of 1973-2011 has been presented above. During 2001, the maximum area is observed to be covered by built up area, followed by agricultural land and waste land, which are 34.88%, 23.11% and 18.01 respectively. In the year 2009 there is an increase in the built up area which is 45.63 and the areas of agricultural land and waste land have been diminished, which are17.38% and 16.31% respectively. Thus the maximum area is occupied by built up area only. Even in year 2011 there is an increase in the built up area to 51.82 %.The constant increase in the built up area in PCMC area indicates speed of urbanization. The increase in industrialization and urbanization has affected land use, in years 2001, 2009 and 2011.

3.1 Economic profile of Pimpri chinchwad:

Pimpri-Chinchwad along with Pune is one of the biggest industrial centers in Asia. Automobiles industries, information and technology, real estate, manufacturing and exports, research centers and education were the fast growing sectors in Pimpri-Chinchwad. Many industries and companies prefer Pune and Pimpri-Chinchwad Municipal Corporation region as their most favorite location for business investments Industries in Pimpri-Chinchwad The IT and Information Technology enabled Services (ITES) sectors has changed the profile and economic conditions of Pimpri-Chinchwad Real estate development: The real estate development in the Pimpri- Chinchwad Municipal Corporation is higher than that of the Pune City. Properties were bought not only for setting up new industries or companies but also for the residential purposes.

3.2 Demarcation of fringe zone and study area:

The fringe zone or the influence zone of the case study area is demarcated by calculating the Urbanization index and scaling it on urban scale. The different settlements or administrative units have their own influence zone which has their circle of influence which varies with the distance and parameter. After the fringe zone is demarcated for the Pimpri-Chinchwad city, study area for the concern research is selected on the basis that it should represent the overall scenario of the region. So villages are selected on that basis. In all 5 villages are selected as the case study area.

3.3 Delineation Approach of Peri-Urban Area: Pimpri-Chinchwad:

A Statistical Approach For delineation of the fringe of pimpri-chinchwad the method used follows these steps

• Calculate urban index for all villages Samples in fringe depending on 16 variables.

• Correlating the variables to find out the relation between any two variables.

• Determining the Scale of Urbanization.

• Determining the suitable methods (Mean, Median & Mode) to delineate fringe area.

• Grading of villages based on the index of urbanization which is to be calculated with the 16 variables The variables are Density, Sex ratio ,Literacy Rate, Decadal Growth, Percentage of Working Population Percentage of Main Worker, Percentage of Marginal Worker, Percentage of Cultivators, Percentage of Agricultural Labor, Percentage of Other Worker, Number of House Hold, Size of House Hold, Distance from city Centre in km, Average Land Value in Thousand/sq.m. Number of BPL families

3.4 Urban Index (UI) Calculation:

Urban index of fringe village is calculated using index values of the town, village and fringe units. Tthe urban index will be:

$UI = (F-V)/ (T-V) \times 100$

where, T, V & F are Index Value of Factor for sample Towns, Villages and Fringe respectively. It is the weighted sum of the sub variables of that variable. Correlation among the variables is seen for sieving out the factors which do not exhibit any marked distributional pattern and have less degree of impact. K. Persons Product Momentum Correlation Coefficient for ungrouped data has been followed. The value of coefficient ranges from -1.00 TO +1.00. A correlation of +1 implies that one variable increases or decreases exactly with other. A correlation of - 1 denotes exact inverse relationship. Zero signifies no relation.

Table 3: Urbanity index calculation of Peri urban area around Pune City

Name of village	Density	ex rat	Literatu re rate	Decadal growth	% of working populati on	% of cultivator	%of agriculture labor	%of other work	No of house hold	Size of house hold	Distance from cc	Avg land value in sq.m	No of BPL Families	Scale Of Urbanity
Jambhe	300	96	74	172	39	21	2	27	348	5	7	13978	60	-31
Marunji-	740	139	71	139	85	7.5	2	31	1145	5	8	12912	170	-26
Gahunje	809	155	30	76	51	6	4	38	863	6	9	15000	100	-22
Dehu	1075	107	80	31	39	7	0.5	19	1381	5	8	10760	130	-62
Chimbali	583	112	74	35	40	11	3	25	1297	6	8	10000	110	-60
T	**5600**	**948**	**89**	**23**	**41**	**5**	**2**	**34**	**7537**	**5**	**14**	**76666**	**9441**	
V	**126**	**129**	**73**	**50**	**76**	**47**	**7**	**22**	**61**	**6**	**15**	**11111**	**6000**	

3.5 Rural Urban Component

In the delineation of the study region, the pimpri chinchwad municipal corporation consists of 632 rural areas. In these areas, 143 villages are in Mulashi Taluka, 181 villages in Maval Taluka and 123 in Haveli Taluka and 185 in Khed taluka. Out of these villages, 5 are selected from different taluka which are at the distance of 13-14 Km from Chinchwad village. [Census data 2011]

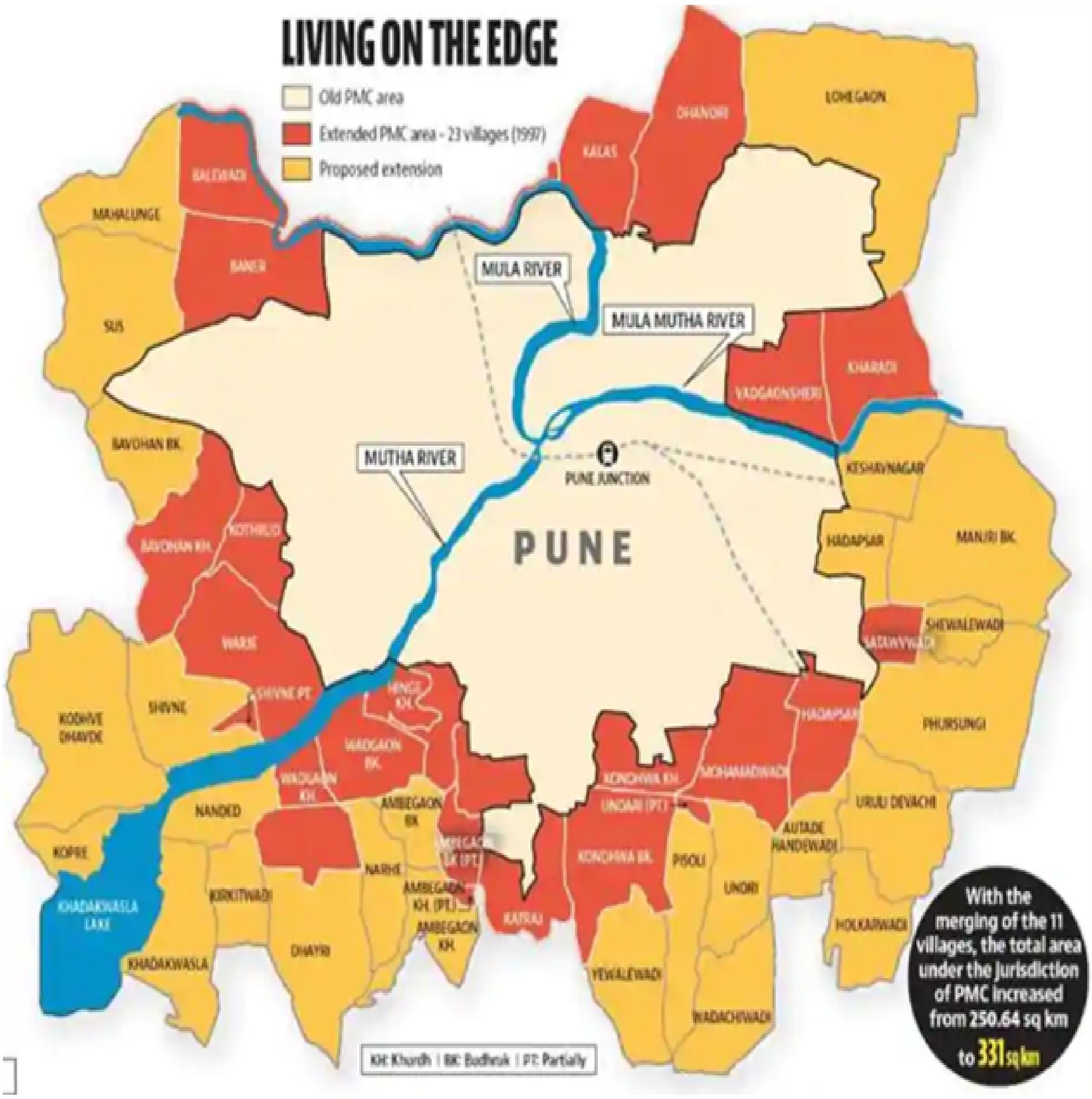


Figure 8: Urban development ,Pune

4 Selection and delineation of Study area Criteria:

Criteria for selecting the sample villages for the study:

- Proximity (Map), within a radius of 10-15 km
- Population Size
- Growth rate
- Density
- % of Non-Agriculture Worker

All these variables were studied from of four different Taluka (Mulashi, Maval ,Haveli and Khed). On the basis of literature review and secondary analysis it has been found that the region covering these villages was urbanizing due to its connectivity, availability of land and vicinity to the core city. As far as the residential development is concerned this urban corridor having highest potential of development Observation: As per data analysis and urbanity index calculations it is observed that Gahunje village is the least developed peri urban area around PCMC. The village is facing issues like electricity outage, water supply shortage, rationing of food grains, lack of health care and education facilities etc.

5 Profile of development:

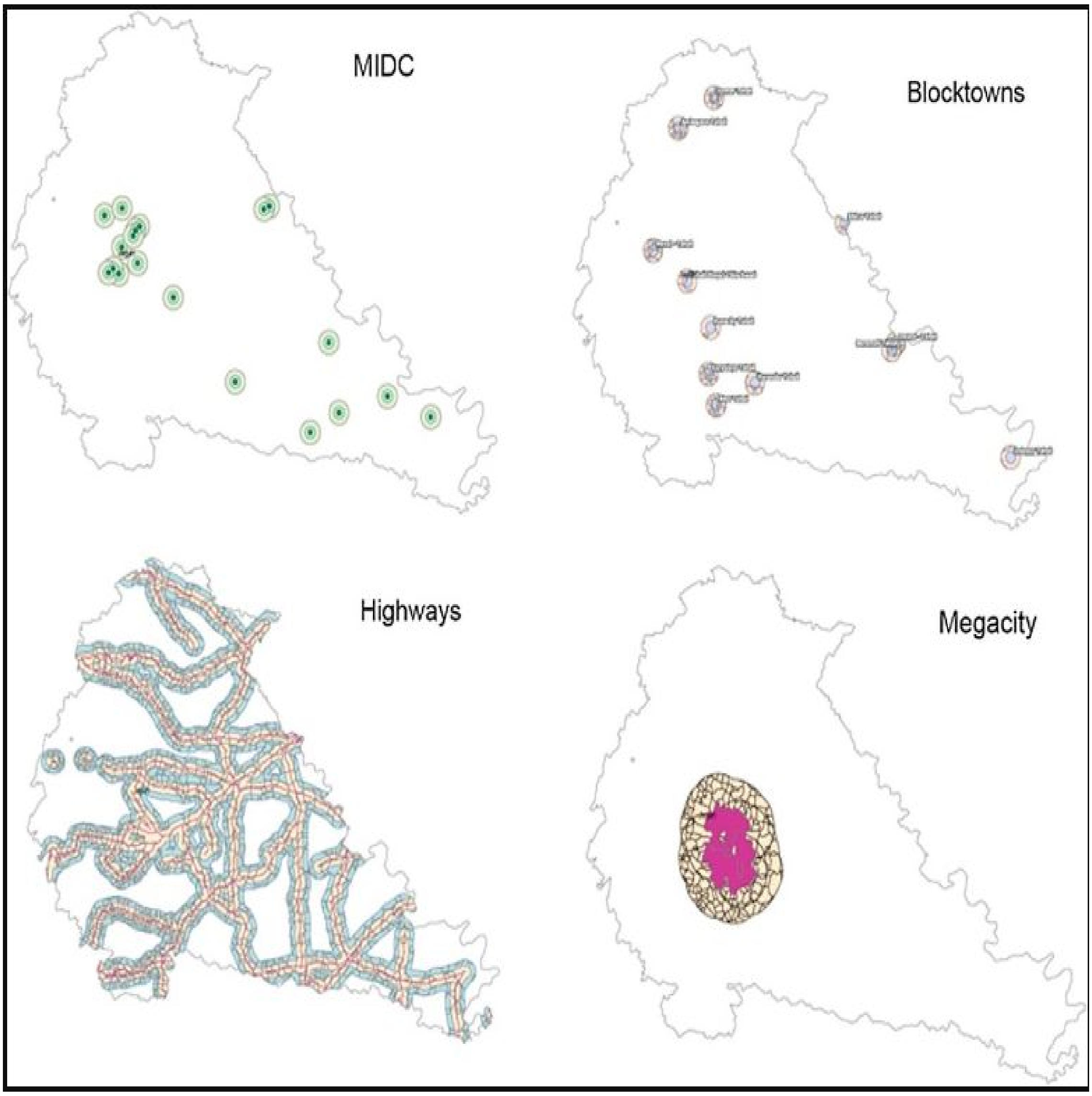


Figure 9: Factors responsible for peri urban growth around city

Highway corridors extend urbanization processes to distant communities. Highway gram panchayats reported that major roads affect real estate values, access to amenities, and location of commercial establishments such as hotel dhaba restaurants, markets, and retail shops (e.g., fabric stores and outlets). These highway patterns are strongest at intersections with secondary roads. Block town (taluka) headquarters concentrate local administrative power, transportation hubs, and market opportunities. Proximity to block town headquarters can have positive implications for village access to government funding and support. Block towns are important secondary nodes for peri-urban development. Cities with over a million in population have transformative effects on nearby settlements. Some villages seek to be annexed by the city, as their growth stresses cannot be accommodated by rural schemes. For example, Pharsungi village sits on the periphery of Pune city and has a staggeringly large population of over 50,000. Gram panchayat members expressed a desire to be part of the Pune Municipal Corporation (PMC) to access funds for improved infrastructure, such as piped water schemes, and relief from urban waste disposal.

6 Conclusions

1. Urban sprawl is encroaching into the the agricultural areas, hence loss of arable lands.
2. Land is sold but not yet developed, that means real estate speculation is high
3. Since there is no master plan/ development plan of the region, unplanned developments are taking place.
4. There is lack of coordination among the administrative bodies.
    - Though the villagers are selling off their land, but they are not selling off their residences, and preferring to travel to the city for basic needs. The social structure of the village is still rural because they are staying there.
    - Construction of houses is taking place on agricultural land without taking proper permission from the collector (NA department), so productive agricultural land is lost.
    - Dumping of solid waste and its partial processing in the existing landfill site is creating continued environmental pollution in the area.
    - A burgeoning population, unprecedented infrastructural growth, waste disposal issues, rapidly changing employment profiles, and decreased cultivation of agricultural land are major types of issues in this peri urban area.
    - The concerns of Gahunje village included electricity outage, water supply shortage, rationing of food grains issues etc. Other problems that are pinching the people are unemployment, MANREGA, and primary health center issues.
    - Health problem is rampant in this village/ locality. People have to travel several kilometers to reach hospital for treatment of patients. Several patients die on the way to hospital.
    - Rationing of food is also an issue at Gahunje, people do not get enough food for their family.
    - Another issue that now a days people at Gahunje are facing is housing issue. People have not enough space for living and 4 to 5 members of family live in one compact room.

7 Recommendations:

Key policy areas for consideration with respect to peri-urban development include the following:

- Prevention: To restrict the city from sprawling and creating fringes along the transportation corridors and city boundary. This can be done by imposing conversion tax and development charges which will be progressive both in distance and time.
- Price signals: should be used in planning of the peri-urban areas.
- Investment : is likely to be highly localized within periurban areas, but this makes sense in that highly nucleated settlement patterns are more efficient (in economic, energy, and fiscal terms), conserve land, and provide better lifestyle opportunities. Public investment, not just for mega-projects such as airports and hi-tech zones, but also for civil infrastructure, can lead and guide peri-urban development, rather than just responding to it, encouraging development of relatively high density nodes.

- Conserve the Agriculture land and rural livelihood by value added farming in the high Agricultural potential areas.

8 Need of planning framework:

In the context of the spatial planning, there is no physical boundary or a limit that can define the two basic phenomena, one of the developed area and another is undeveloped area. Boundaries on the ground and particularly in the context of urban area, doesn't have any influence on the development forces, the market behavior, growth pattern and the pressure on land. There should be appropriate framework and proper hierarchical institutional mechanism, for the development in the urban area. Proper Planning framework and institutional approach is also need for the peri-Urban Villages.

References

[1] "Urban India 2011: Evidence." New Delhi,

[2] “Development Plan, Indore”. Indore Development Plan. Indore: TNCP-Bhopal, 2021

[3] Census 2011, India Stats: Million plus cities in India. Census 2011.

[4] http://www.censusindia.gov.in/2011provresults/paper2/data_files/India2/Table_3_PR_UA_Citiees_1Lakh_and_Ab 2011-10-19

[5] CDP. Indore Development Plan - 2021 (Draft). Indore: TNCP-Bhopal,2021.

[6] Sinha, M.M.P. The Impact of Urbanization on Land Use in the Rural Urban Fringe: A case of patna. Patna: Concept Publishing Company,1980.

[7] Ramachandran, R. "The Rural-Urban Fringe." Ramachandran, R.Urbanization and Urban System in India. Oxford University Press, 1989.297.

[8] Bentinck, Johan. Unruly Urbnisaiton on Delhi's fringe. Netherland geographical Studies, 2000.

[9] McGee, T. The Extend Metropolis: Settlement Transition in Asia.University of Hawaii, 1991.

[10] Rao, M. Urbanisation and Social Change. Orient Longmans Ltd., 1970.

[11] Rao, P. Some Aspects of Urbanisation. Madras: Prasaranga, May,1971.

[12] "Rural - Urban distribution of Population (Provisional)." 2011.